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Waterways Experiment Station

Comparison of Idealization Schemes for One-Dimensional Site Response Analysis

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Prepared for U.S. Department of Energy

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Prepared for U.S. Department of Energy
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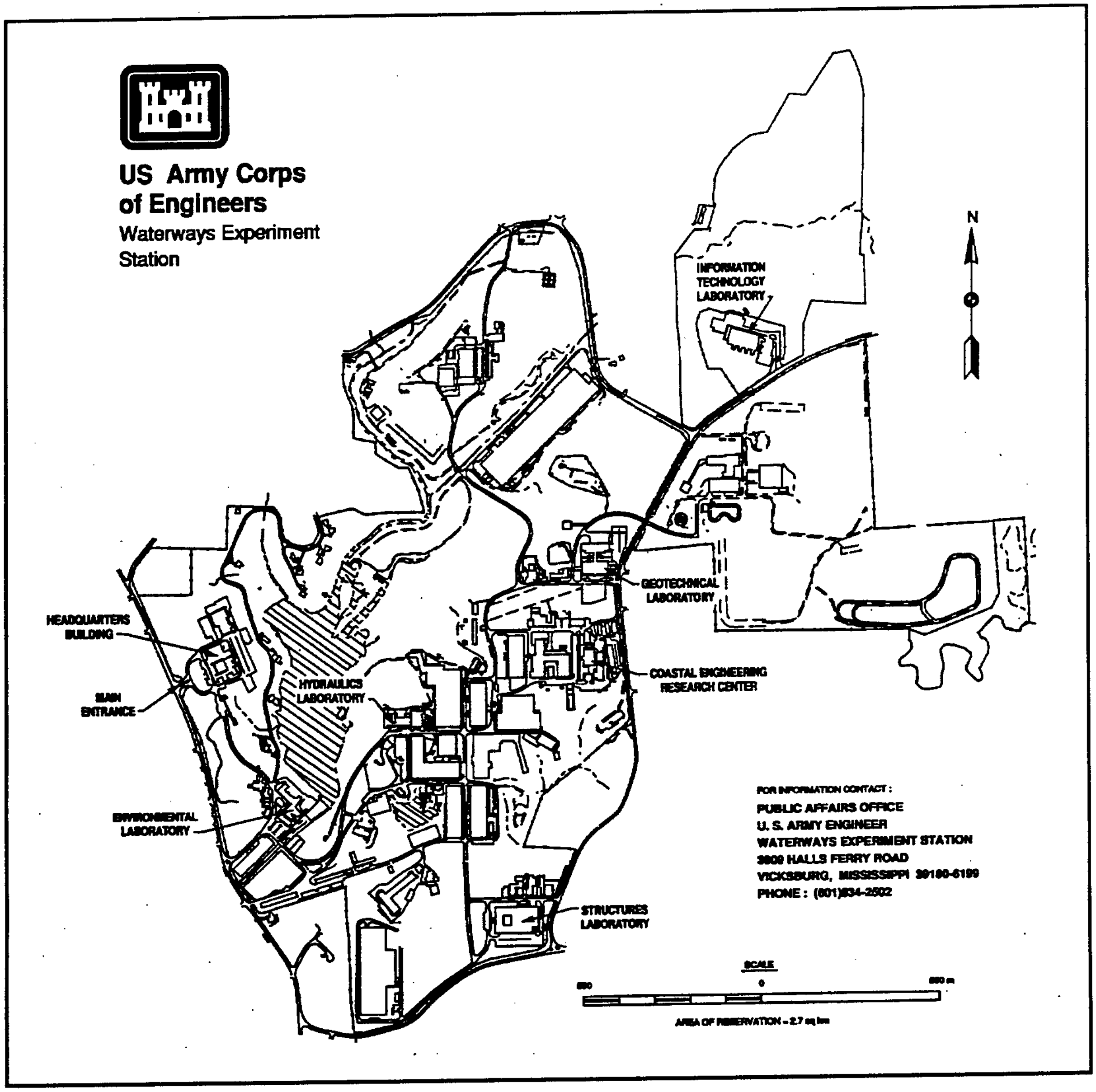

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Preface

This report summarizes a research study conducted by the U.S. Army Engineer Waterways Experiment Station (WES) for the Department of Energy (DOE) through the U.S. Army Engineer District, Charleston, under MIPR Number CESAC-RM-93-82. The USACE Savannah River Site (SRS) Project Manager was Mr. David Sanders (CESAC-SR), and the DOE monitor was Mr. Brent Gutierrez (AMEP-PSD-CB).

The Principal Investigators were Dr. David W. Sykora and Prof. Carl J. Costantino. Dr. Sykora began this study while employed in the Earthquake Engineering and Seismology Branch (EESB), Earthquake Engineering and Geosciences Division (EEGD), Geotechnical Laboratory (GL), WES. Prof. Costantino is a faculty member of the Department of Civil Engineering, City College of New York. Dr. Sykora was assisted by Mr. Donald E. Yule, Ms. Wanda I. Cameron, and Ms. Jennifer J. Smith, WES; Prof. Costantino was assisted by Dr. Ernest Heymsfield, City College of New York. Also assisting in the study was Dr. Norman A. Abrahamson, Engineering Seismology Consultant, under contract to WES. Dr. Abrahamson was tasked to select appropriate earthquake records for general parametric comparisons and to evaluate measured variabilities in ground motions over short distances. His contract report (Abrahamson 1993) forms the first part of chapter 5. Dr. Mary Ellen Hynes was the Chief, EESB, during this study.

The following institutions made the seismic strong motion data available: the Institute of Industrial Science, University of Tokyo, provided the Chiba Array data; the U.S. Geological Survey provided the USGS Parkfield, Imperial Valley Differential, Hollister Differential, and Coalinga array data; the University of California, Santa Cruz, provided the UCSC ZAYA array data.

Overall direction at WES was provided by Dr. A. G. Franklin, Chief, EEGD, and Dr. William F. Marcuson III, Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Conversion Factors, non-SI to SI (Metric) Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	Abbreviation	By	To Obtain
acres	-	0.4047	square kilometers
feet	ft	0.3048	meters
inches	in.	2.540	centimeters
miles (US statute)	mis.	1.609	kilometers
pounds (mass) per cubic foot	pcf	16.018	kilograms per cubic meter
pounds (mass) per square foot	psf	4.882	kilograms per square meter
tons (mass) per square foot	tsf	9,765.	kilograms per square meter
pounds (force) per inch	psi	175.1	newtons per meter

1 Introduction

Site response analyses have been completed at a number of large Department of Energy (DOE) project sites that are being analyzed for seismic safety. The objective of a site response analysis is to estimate the free-field ground shaking during an earthquake, that is shaking at sites that does not include effects caused by proximity to structures or topographic features, for a specific hazard level and set of site conditions. The requisite components for a site response analysis are: one or more design earthquake events with representative earthquake record(s), an idealization of the soil-rock system at the site of interest, and a scheme to generate response solutions to simplified assumed wave fields. Normally, the free-field ground response is presented in terms of either response spectra or the variation of acceleration or velocity with time. The alternative to conducting a site-specific response analysis is to use general guidelines recommended by Kennedy et al. (1990).

Of the three requisite components listed, the effects of different idealization procedures on calculated response seem to be the least studied, although different procedures are being used for DOE projects. Therefore, the primary objective of this study was to compare two idealization schemes recently used for the calculation of horizontal site response using one-dimensional (1-D) solution methods.

The two procedures are termed the "individual column" and the "best-estimate" methods. The individual column method involves analyzing a number of individual soil columns, each representing a unique set of soil parameters at a specific subsite (measurement) location. This method has been primarily used for studies at the Paducah and Portsmouth Diffusion Plants (Sykora and Davis 1993; Sykora and Davis 1993). The best-estimate method involves averaging information from each specific subsite location into one idealized representation. Use of the best-estimate method typically involves a parametric analysis of model inputs, most

importantly, shear modulus. Recommendations by the Nuclear Regulatory Commission (1989) and the American Society of Civil Engineers (1986) were used to define upper- and lower-bounds for shear modulus. This method has been used for the K-Area of the Savannah River Site (Costantino et al. 1991).

It is important to differentiate between characterization and idealization of a project site. Site characterization is the process of identifying the distribution of materials and their respective physical and engineering properties. Idealization is a subsequent process of simplifying the geometry and variation of materials for numerical analysis. Idealization may be significantly affected by the requirements and limitations of the computer model (e.g., dimensionality, number of layers, etc.). This study addresses idealization schemes which assumes that the project site has been suitably characterized.

The two idealization methods were compared using "soil column" idealizations specific to two DOE sites and one-dimensional (1-D), frequency-domain site response computer codes. The soil parameters were measured at the DOE Portsmouth Gaseous Diffusion Plant (PORTS) and at several sites at the DOE Savannah River Site (SRS). Despite the use of site-specific soil data, the results of this study are **not** intended to supersede previously-reported results for either project. In fact, the earthquake records used with the PORTS soil data are generic for the western U.S. and, therefore, are not related to the actual earthquake hazard. The study of soil data from SRS with a site-specific earthquake record is best characterized as being an extension of parametric analyses already conducted and reported by others.

The results from the two idealization methods are also compared with measured variations of ground response during earthquakes. The use of an average column alone would presume that the response over the whole site is uniform. Measured ground response indicates that significant variations can occur over short distances.

It should be noted that one-dimensional codes may not be appropriate for sites where valley or other topographic effects are expected to be important (Electric Power Research Institute 1991). Ultimately, the effects of idealization in complex subsurface conditions should be examined with multi-dimensional analyses.

This report begins by briefly describing the components of an earthquake site response analysis in Chapter 2. Then, the effects of idealization schemes are evaluated by examining data and results of site response calculations from a shallow site (PORTS) in Chapter 3 and a deep site (SRS) in Chapter 4. In Chapter 5, the measured variation of ground response is examined. This report closes with Chapter 6, summary and conclusions, and recommended procedures in Chapter 7.

2 One-Dimensional Site Response Analysis

The three basic components of a site response analysis - earthquake records, site idealization, and calculation methods - are described below. Earthquake records and response calculations are described in some detail whereas the effect of idealization schemes is introduced and discussed in detail in Chapters 3 and 4.

Earthquake Records

Four earthquake motions were used for this study. Three measured earthquake records representing unique combinations of earthquake magnitude and epicentral distance were chosen for the analysis of the shallow site described in Chapter 3. A site-specific synthetic record was used for the analysis of the deep site described in Chapter 4. Descriptions of each set are made below.

Measured Records

Three measured records were selected based on a set of magnitude and distance criteria for lower levels of ground shaking listed in Table 2.1. Other selection criteria were that the record should correspond to rock or hard soil conditions and that the spectra have a relatively broad band of energy. The records were not intended to be specific to either the eastern U.S. (EUS) or the western U.S. (WUS). The records were selected by searching an up-to-date data base of strong motion records for rock sites (Sadigh et al. 1993) to find the records whose average horizontal spectrum was similar to a smoothed empirical spectral shape for the desired magnitude and distance ranges. The records chosen and their characteristics are given in Table 2.2.

Table 2.1

Ranges on Magnitude and Distance Imposed for Search

Name	Magnitude Range (M_L)	Distance Range (km)
M5	5.3 - 5.9	0 - 20
M6	6.0 - 6.5	20 - 50
M7	6.5 - 7.5	40 - 100

Table 2.2

Earthquake Records Selected

Name	Earthquake	Year	Record	M_L	Distance (km)	a_{max} (g)
M5	Coyote Lake	1979	Gilroy #1	5.7	10	0.12
M6	Morgan Hill	1984	Lick Observ.	6.2	44	0.08
M7	Loma Prieta	1989	APEEL 3E	7.0	53	0.08

The horizontal 1 components of the M5 and M7 events and the horizontal 2 component of the M6 event were band-pass filtered and baseline corrected using a high order polynomial curve fitting procedure. The variation of acceleration response and response spectra at 5 percent system damping for these three components of the selected records are presented in Figs. 2.1 through 2.3. Note that these records do not contain the high frequency content ($f >$ 15 Hz) that is expected for EUS hard rock sites. Unfortunately, there is little strong motion data available from EUS hard rock sites. The peak acceleration was 0.12 g for the M5 event and 0.08 g for the other two events.

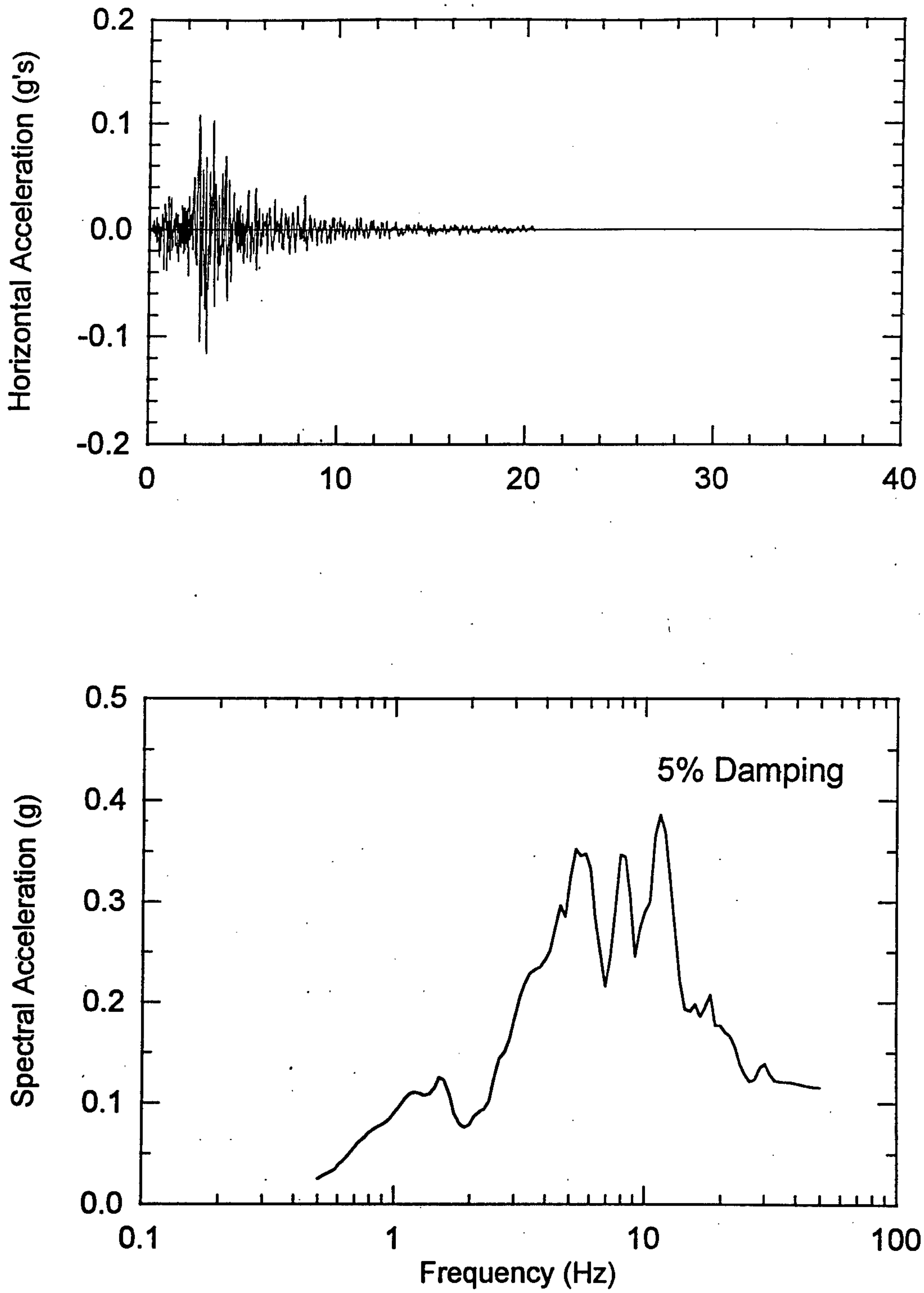

Figure 2.1 Acceleration record and response spectra for horizontal 1 component at Gilroy #1 during 1979 Coyote Lake earthquake (M5 event)

m5_eq.grf

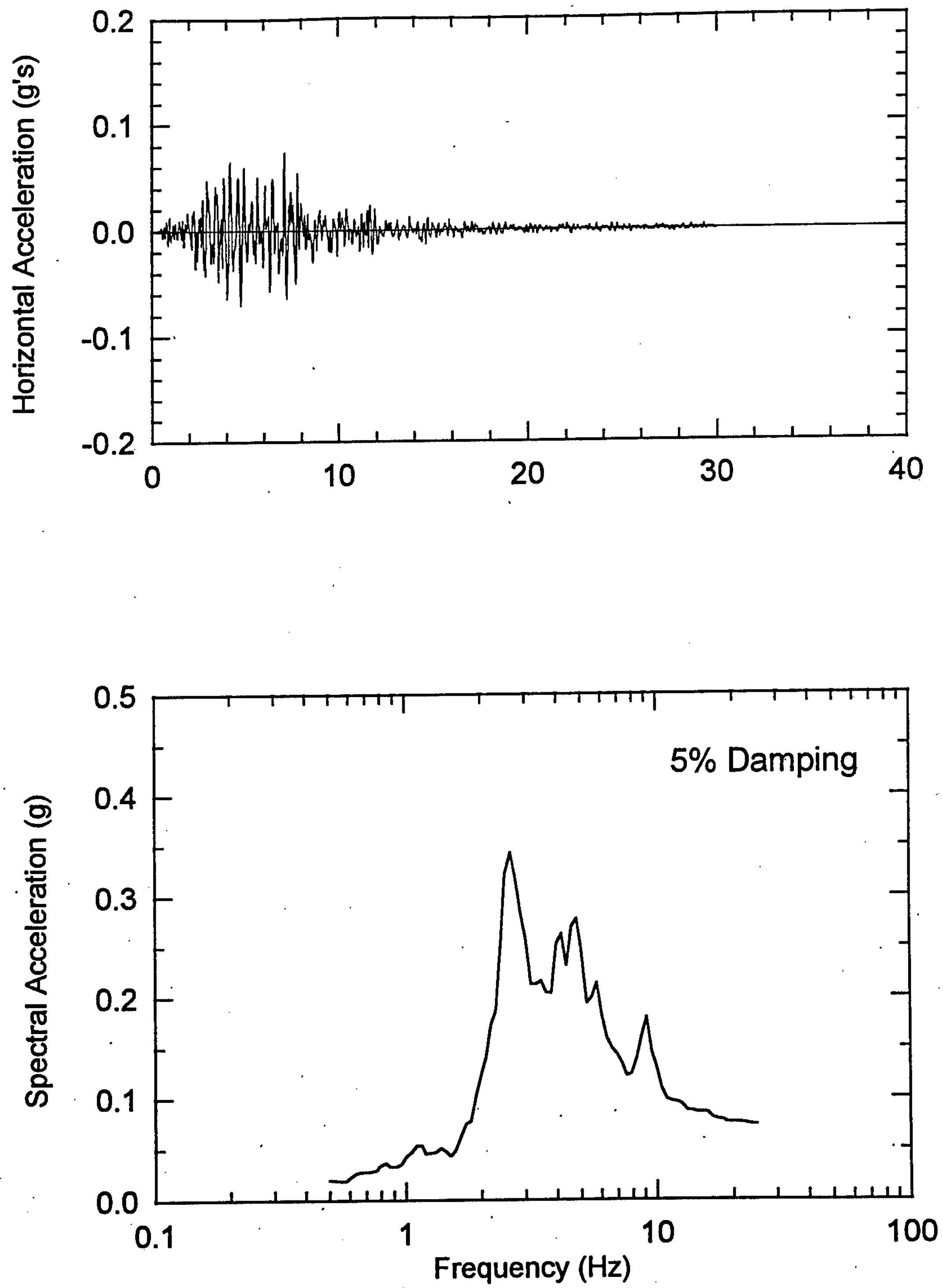

Figure 2.2 Acceleration record and response spectra for horizontal 2 component at Lick Observatory during 1984 Morgan Hill earthquake (M6 event)

m6_eq.grf

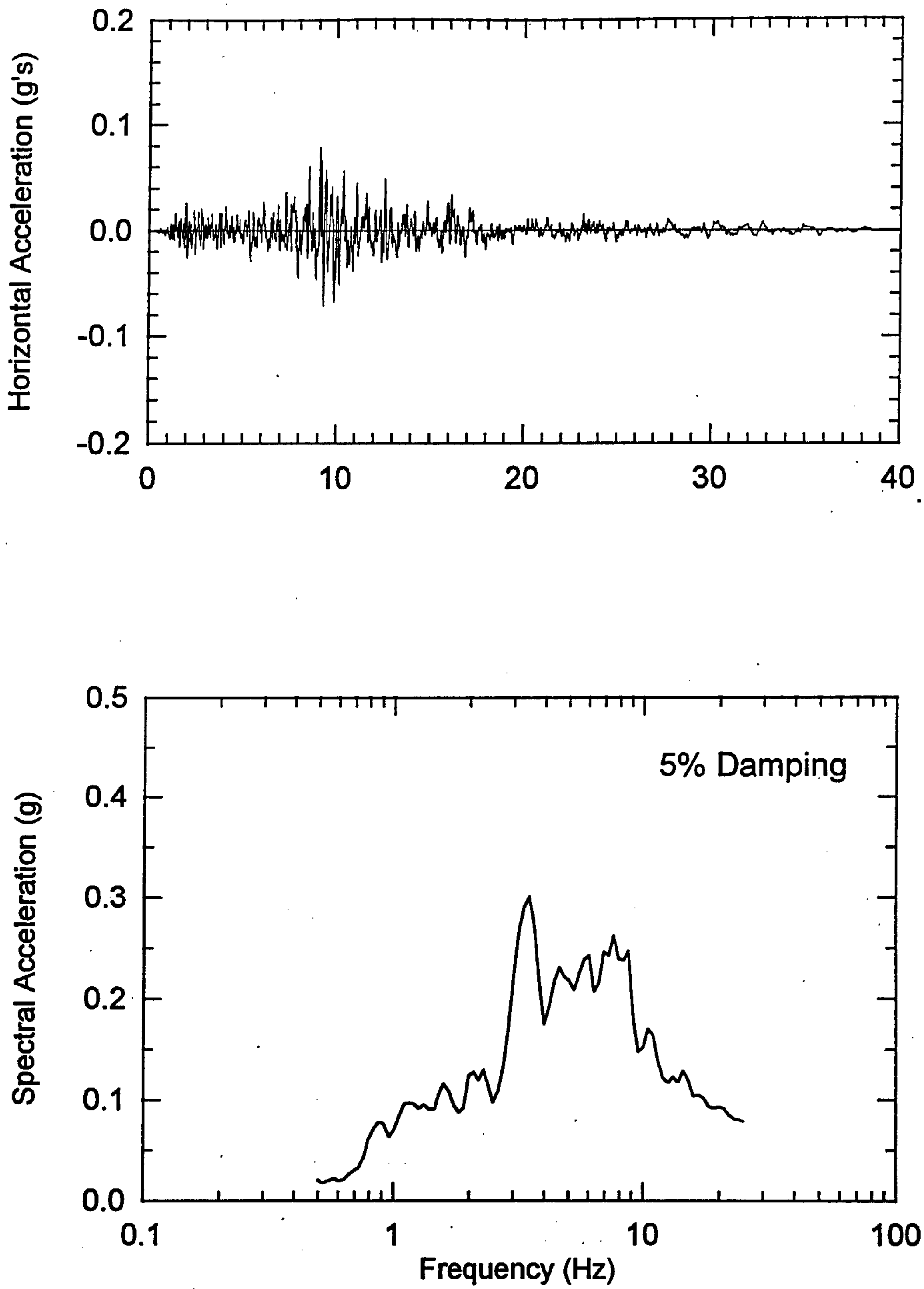

Figure 2.3 Acceleration record and response spectra for horizontal 1 component at APEEL 3E during 1989 Loma Prieta earthquake (M7 event)

m7_eq.grf

Savannah River Site

The synthetic earthquake record used in this study was developed by fitting a time history to a site specific response spectrum developed for the H-Area of the Savannah River Site. The response spectrum was developed using a Ou and Herrmann (1990) attenuation model appropriate for the SEUS for a Charleston 1886 source with $M_w = 7.5$ at a distance of 120 km and a stress drop of 100 bars. The acceleration record and response spectra for this synthetic earthquake (M7.5 event) are shown in Figure 2.4. The peak acceleration for this record is 0.055 g.

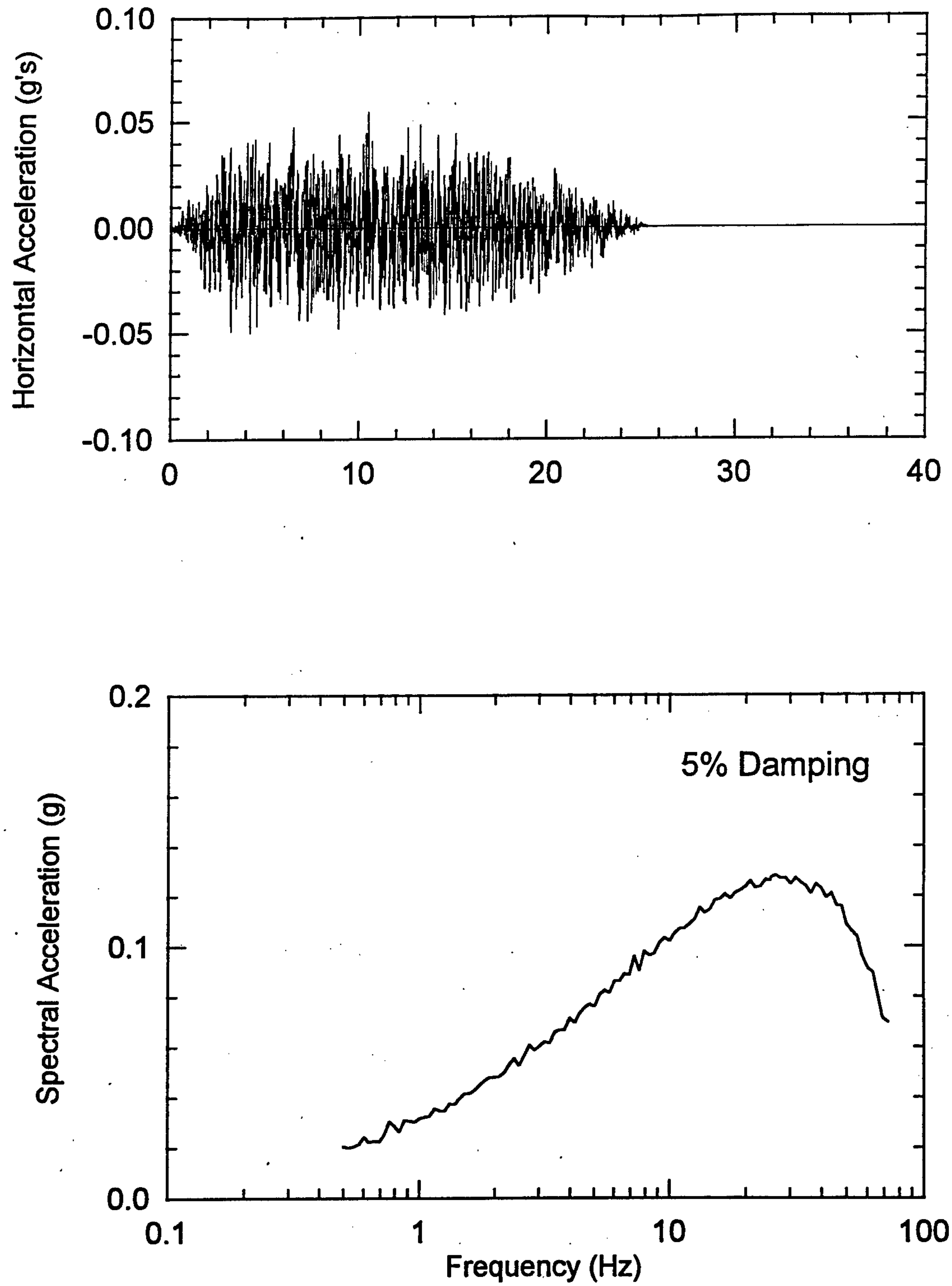

Figure 2.4 Acceleration record and response spectra for synthetic earthquake (M7.5 event) used at ITP site

itp_eq.grf

Site Idealization

Shear modulus and its variation with depth is the key soil property in a site response analysis. A soil column idealization for site response calculations is a representation of the vertical distribution of material property types and shear stiffness with an appropriate number of layers over which the material type and shear modulus is assumed to be constant. Shear modulus is derived from two measured quantities--shear wave velocity and mass density, both of which are likely to vary independently with depth. Furthermore, the variation of shear modulus and damping ratio with shear strain are generally associated with material type and soil plasticity which may also vary with depth. Therefore, the process of deriving an idealized soil column includes comparing the profiles of shear modulus (or its two components, shear wave velocity and mass density), material type, and plasticity index.

Individual Columns

An individual column is typically intended to represent soil conditions within a small plan area and, therefore, is typically derived from information from a single boring or a small array of adjacent borings or soundings. Because this data is derived from a small spatial area, the idealization process is constrained to match this local character. In developing these individual columns, each input element can be carefully considered and the resulting column is a close representation for this area. To represent the response over the project site, the response calculated for each individual column are combined and the resulting range evaluated. Therefore, some of the inherent variability of the site response, that are due to variations of site conditions and soil properties is explicitly included.

Best-estimate Column

A best-estimate soil column is typically generated for a site in which

relatively uniform conditions exist over the entire site or a relatively large portion of the site. Since the best-estimate (or average) soil column is generated from information from a number of borings or soundings, this column may not represent any one area very well. To account for the potential variability in soil stiffness and density over the site, responses are calculated using upper- and lower-bound values for the shear wave velocity of each soil layer. The variability in thickness of individual soil layers is typically included in generating the best-estimate, upper- and lower-bound soil columns. The effects of variability in depth to bedrock, however, is usually not explicitly incorporated in these variations.

Establishing a procedure of generating a best-estimate soil column for sites with major variations in depth of the soil column is more problematic. Therefore, for sites with major changes in soil column thickness, more than one best-estimate soil column together with its upper- and lower-bound values may be used in site response studies to account for significant changes in potential frequency shifts in peaks of the surface response spectrum.

Calculation Methods

Two one-dimensional wave propagation computer codes were used in this study: *SHAKE* and *CARES*. *SHAKE* was used to analyze individual column idealization schemes and *CARES* was used to analyze best-estimate column schemes. Comparisons of calculated response spectra from these two computer codes indicated that the effect of using two codes produced negligible differences . Each computer code is described briefly below.

SHAKE

SHAKE (Schnabel, Lysmer, and Seed 1972) was developed to calculate the horizontal response caused by an earthquake at any depth of a soil profile. The

methodology and algorithms incorporated in the program are fairly simple and straight-forward and quite adequate for the purpose intended as clearly evident based on favorable comparisons with measured response (e.g., Seed et al. 1987; and Seed, Dickenson, and Idriss 1991) and peer review and acceptance of its prolific use. The simplicity associated with *SHAKE* is attributed to some basic assumptions regarding the cyclic behavior of materials and geometry of the problem. The basic assumptions of importance to this study are:

a. Soil layers are horizontal and extend to "infinity";

b. Ground surface is level;

c. Each soil layer is completely defined by the shear modulus and damping ratio as a function of strain, the thickness, and unit weight;

d. The cyclic behavior of each soil (and base rock) is represented by the equivalent-linear representation model;

e. The material damping is frequency independent;

f. The incident earthquake motions are spatially uniform, horizontally-polarized shear waves propagating vertically. (1-D wave propagation model)

The one-dimensional wave equation model (Kanai 1951) was used to develop *SHAKE*. This model has proven to be effective despite the simplicity and number of assumptions involved. The solution algorithm involves the complex response technique and the Fast Fourier Transform (Cooley and Tukey 1965). The general formulation of the wave equation is not limited to horizontally-polarized shear wave motion; the equation can also be solved for the vertical propagation of compression waves.

In general, soil is a non-linear material that exhibits hysteretic behavior

under cyclic loading. The equivalent-linear representation incorporated in *SHAKE* is linear but accounts for the dependency of moduli on shear strain. This method (proposed by Seed and Idriss, 1970) is widely used in geotechnical earthquake engineering studies.

The basic components of the equivalent-linear method are the maximum shear modulus, G_{max}, moist unit weight, and ratio of critical damping, β . G_{max} can be calculated from low-strain seismic shear wave velocity using:

$$G_{max} = \rho \, V_S^2 \tag{1}$$

where

ρ = mass density (moist unit weight / gravitational constant)

V_s = shear wave velocity

or from the shear modulus coefficient, $(K_2)_{max}$, which is defined by Seed and Idriss (1970):

$$G_{max} = 1000 \, (K_2)_{max} \, (\sigma_m')^{0.5} \tag{2}$$

where

σ_m' = mean effective stress, in psf

G_{max} is in psf

Shear wave velocities (equation 1) were used exclusively for this study.

At shear strains generally greater than about 10^{-4} percent, the stiffness decreases to values less than G_{max} . The equivalent-linear method uses secant shear moduli that are adjusted during each iteration to account for this. Damping is input by using complex moduli, G^*, which is independent of frequency:

$$G^{*} = G\,(1-2\beta^{2}+2i\beta\sqrt{1-\beta^{2}}) \tag{3}$$

where

$$i = \sqrt{-1}$$

Damping increases as shear strain increases. The character of these functions of strain was first addressed in studies by Hardin and Drnevich (1972), Seed and Idriss (1970), and Schnabel (1973). Later studies include: Zen and Higuchi (1984), Seed et al. (1986), Sun, Golesorkhi, and Seed (1988), and Vucetic and Dobry (1991).

CARES

CARES (Costantino, et al, 1993) is a software package that treats several different aspects of the seismic response problem, namely, (1) a motion generation component that develops acceleration time histories to match target response spectra for specified magnitude events, (2) a free-field component that treats the one-dimensional shear wave propagation upward through a given soil column, and (3) a structural response component that treats the soil-structure interaction to generate seismic response of structural models. In the free-field segment of the code, the soil column response is determined in a fashion similar to *SHAKE*, with some relatively minor variations in computations, additions to the soil degradation models and specific options appended to the code to provide greater versatility. Comparisons of results from the two codes for the same problem indicates similar results.

Sensitivity Analysis

A sensitivity analysis is an important aspect of a site response analysis. All input values may be considered in this aspect, but in most studies the effect of varying shear modulus usually is the most important.

Both the Standard Review Plan (SRP) of the Nuclear Regulatory Commission (NRC, 1989), and seismic guidelines for nuclear facilities by the American Society of Civil Engineers (ASCE, 1986), have specific recommendations for varying shear modulus in soils for dynamic response calculations. The NRC criteria were considered for this study and are:

Upper bound:	$G^{ub} = 2.0\ G_{max}$	or	$V_s^{ub} = 1.414\ V_s$
Lower bound:	$G^{lb} = 0.5\ G_{max}$	or	$V_s^{lb} = 0.707\ V_s$

Presentation of Results

The primary means used to measure the differences between the two idealization schemes was to compare plots of 5% damped response spectra for surface motions. The ratio of soil to rock spectral acceleration is also presented for each case as a measure of site amplification. Comparisons are made separately between effects of upper and lower bounds to shear modulus on the results versus the combined effect of using the bounds plus enveloping the combined results. In this way, the two effects can be isolated and the effect of idealization examined.

Results are presented for frequencies greater than 0.5 Hz and lower than the Nyquist frequency based on the experience of investigators who have compared calculated free-field response with measured response from major earthquakes. These comparisons suggest that one-dimensional calculations be limited to frequencies greater than 0.5 Hz. At lower frequencies, the motions are likely to be significantly affected by two-dimensional effects and surface wave energy, which are not well represented with one-dimensional codes.

3 Shallow Site Evaluation

An evaluation of the two idealization methods was made using data from a large shallow soil site the Portsmouth Gaseous Diffusion Plant (PORTS). Shallow sites are generally expected to respond at higher frequencies of motion. The thickness of overburden and range of shear wave velocities were found to vary considerably. Nine columns, derived from nine profiles of measured V_s , were used for the analysis. The three earthquake motions, corresponding to different combinations of magnitude and distance from the fault, were applied with peak horizontal accelerations ranging from 0.08 to 0.12 g.

Site Description

PORTS is located about 20 miles north of Portsmouth, Ohio, about 2 to 3 miles east of the Scioto River. The site is located in the Appalachian Plateau Province, within the boundary of the pre-glacial Portsmouth River Valley. This valley is about 1 mile wide (in an east-west direction) at PORTS with an average surface elevation of about 670 ft and topographic relief generally less than 10 feet. The valley is generally well drained, with overall drainage to the south.

The plant consists of administration and numerous plant buildings and facilities used in the enrichment of uranium. A general site map is shown in Figure 3.1. The overall plant area is about 1,400 acres in size and was leveled prior to construction in the 1950's.

For purposes of this study, geotechnical investigations performed in the late 1970's for a proposed addition to the plant were used to develop individual soil columns. This portion of the site, corresponding to about 300 acres in size, lies along the eastern margin of the overall plant area.

Site Characterization

Soils consist primarily of Pleistocene-age lacustrine deposits of silt and clay and are sometimes underlain by Pleistocene-age alluvial deposits. Holocene-age alluvium exists in remnant stream channels. Fill exists at several locations in the plant area, but not in the area idealized for this study. The thickness of soil within the area of the plant addition varies between 20 and 61 ft, generally thinning toward the east and northeast margins (corresponding to the edge of the valley). Bedrock consists of Sudbury Shale overlying a much harder sandstone. Up to 60 ft of overburden may have been removed by natural processes during Holocene time leaving moderately overconsolidated materials.

Extensive geotechnical investigations and laboratory tests (Taylor et al. 1977) were conducted for the proposed plant addition, including the measurement of shear wave velocity at nine sites using the seismic crosshole method (Curro and Marcuson 1978) at locations shown on Figure 3.1. The closest spacing between the sites is about 300 ft; the farthest distance between sites is about 5,000 ft.

The range of shear wave velocities developed from crosshole measurements in the area of the plant addition is shown in Figure 3.2. At shallow depths, the shear wave velocity ranges from as low as 235 fps to about 1,000 fps and generally increases with depth. The larger velocities near the base of each column represent denser sand and gravel deposits. These velocities may also have been influenced by refracted waves through higher-velocity bedrock.

Site Idealization

The nine idealized individual columns are presented in Figures 3.3 through 3.7. Between three and eight layers were used to define the nine columns with layer thicknesses ranging from less than 2 ft to over 20 ft. The nine soil columns vary in total depth from 20 ft to over 60 ft. A comparison between shear wave velocities for the idealized profiles and the range of measured values is shown in Figure 3.8. The idealized profiles bracket the range fairly well above depths of 22 ft and below depths

of 33 ft. Details of the best-estimate column are shown in Figure 3.9. A comparison between the profile of shear wave velocity for the best-estimate column and the range of measured shear wave velocities is shown in Figure 3.10. Note that the depth of the best-estimate column was selected as 42 ft, an average value for the range of total depths at the site. However, this single column would not be able to capture the site effects from the full range of depths of the original nine soil columns.

The variation of normalized shear modulus and damping ratio with shear strain used for the materials idealized as shown in Figures 3.3 through 3.7 are compared in Figure 3.11. Material 1 represent gravel, materials 2 and 3 represent sand, and materials 4 through 8 are clayey soils differentiated using proposed relationships by Sun et al. (1988)

Analysis

The spectral accelerations and spectral ratio (soil to rock response) calculated for each soil column are presented in Figures 3.12 through 3.14 for the M5, M6, M7 events, respectively. Also shown are the spectra corresponding to the rock (outcrop) motion used for input. The influence of the input ground motion is apparent in Figures 3.12 through 3.14. The peak spectral accelerations for each are nearly equal but the frequencies at which the peaks occur vary considerably. For the M5 event, the range of spectral accelerations is fairly narrow with peaks at about 6 and 11 Hz. The range for the spectral ratio is generally wider. For the M6 event, the range of spectral accelerations is also narrow and several of the columns have peaks at about 2.5 and 4 Hz. The range for the spectral ratio is much wider with one response (Site 7: a short, stiff site) being noticeably different than the rest. For the M7 event, again the range of spectral accelerations is fairly narrow with several peaks at a frequency slightly above 3 Hz. The range of spectral ratios is again fairly wide with the highest value at about 3 Hz.

The results of site response calculations using the best-estimate column are compared with the range calculated from individual columns in Figures 3.15 through 3.17 for the M5, M6, and M7 events, respectively. Also shown are the results using

the upper- and lower-bounds for shear modulus. For the M5 event, the peak spectra acceleration for the best-estimate column is about 20 percent less than the peak spectra from the range of individual columns and at a slightly lower frequency. However, the upper bound formed from the collection of the three best-estimate column responses is very similar to the upper bound formed from the individual columns. A comparison based on the spectra ratio show similar results. The upper bound response at frequencies greater than 10 Hz is formed from the best-estimate column, not from either the upper- or lower-bound profile.

Similar findings were observed for results from the M6 and M7 events. The response for the best-estimate column matches well with the peaks between 4 and 5 Hz from the individual columns but significantly underestimates the peak spectral acceleration between 2.5 and 3.5 Hz. The collection of responses for the best-estimate column and the best-estimate column with the lower- and upper-bound of velocities matches well with the upper bound from the individual columns. These comparisons indicate that the single column spectrum may miss important peaks of the envelope spectrum found from the results of the individual soil columns, as would be expected.

The results using the two idealization methods are also compared in Figures 3.18 through 3.20 by enveloping the best-estimate, upper-bound, and lower-bound shear modulus columns. The enveloping process is intended to include potential variations in frequency and peak spectral accelerations as properties of the soil column vary over their possible range). The comparisons shown on Figures 3.18 through 3.20 suggest that enveloping is an important component to using a best-estimate column to represent the range of potential site response.

For the case where significant variations in depth of the column exist over the site, one objective of the single column approach is to capture the potential variability in site response due to variability in soil properties. These results indicate that if significant variability in column depth is encountered, several "average" soil columns should be used to ensure that the effects of depth of the soil column on the fundamental period of the column is captured.

The parametric bounds to shear modulus for the case of the single average soil column were also used with a Monte Carlo log-normal simulation of the shear modulus profile using constant layer thicknesses. The resulting profiles for the 5th, 50th, and 95th percentiles of shear modulus after 1,000 simulation cycles are shown in Figure 3.21. The depth of the soil column was fixed at the value of the average soil column mentioned above. The resulting spectra at these percentile levels using the M7 event are presented in Figure 3.22. These can be compared with the simple means of applying upper and lower bounds to the best-estimate column presented previously in Figure 3.17. The most noticeable difference is that the statistical analysis of results from the 1,000 Monte Carlo simulations better defines the peaks of the computed spectra as compared to the results of the nine individual soil columns even for this case where a single depth of column was used in the simulations. It should be noted that the use of the nine individual soil columns with best-estimate soil properties defined in each case may not appropriately capture potential variability in surface spectra since the impact of variability in local site properties has not been addressed.

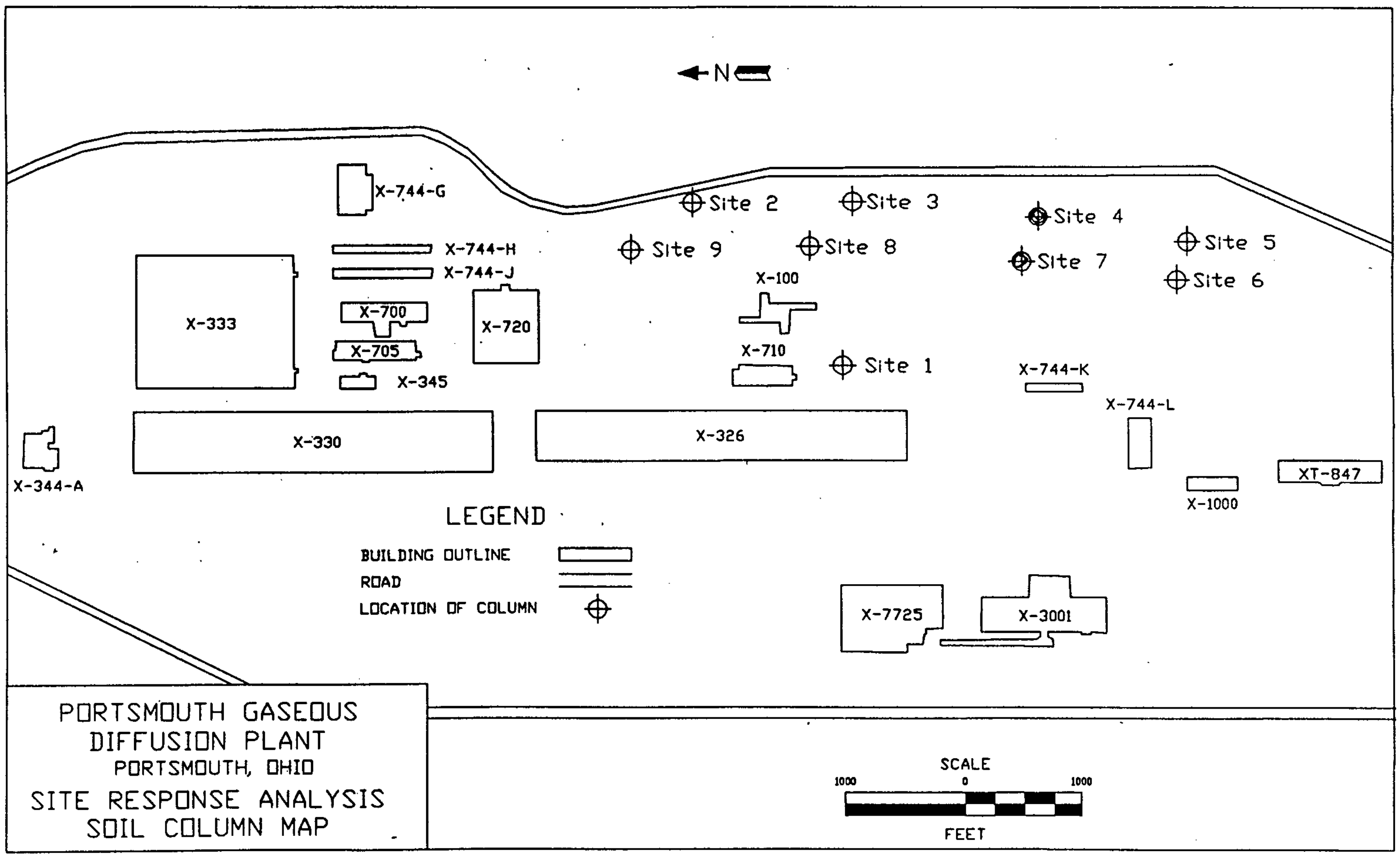

Figure 3.1 Site Plan of Portsmouth Gaseous Diffusion Plant (PORTS) showing locations of sites used for earthquake response evaluations

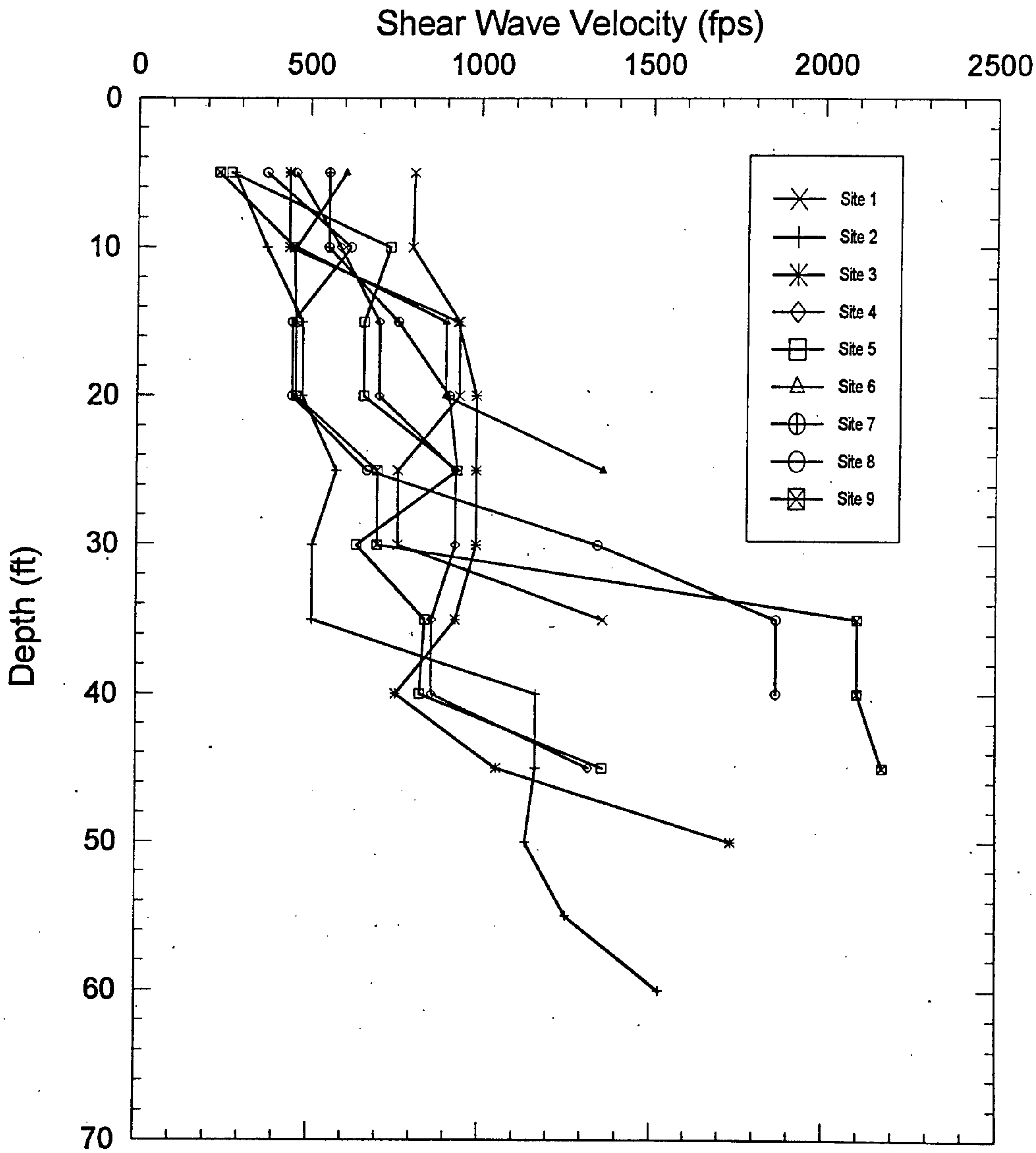

Figure 3.2 Variation in shear wave velocity with depth for shallow sites at PORTS

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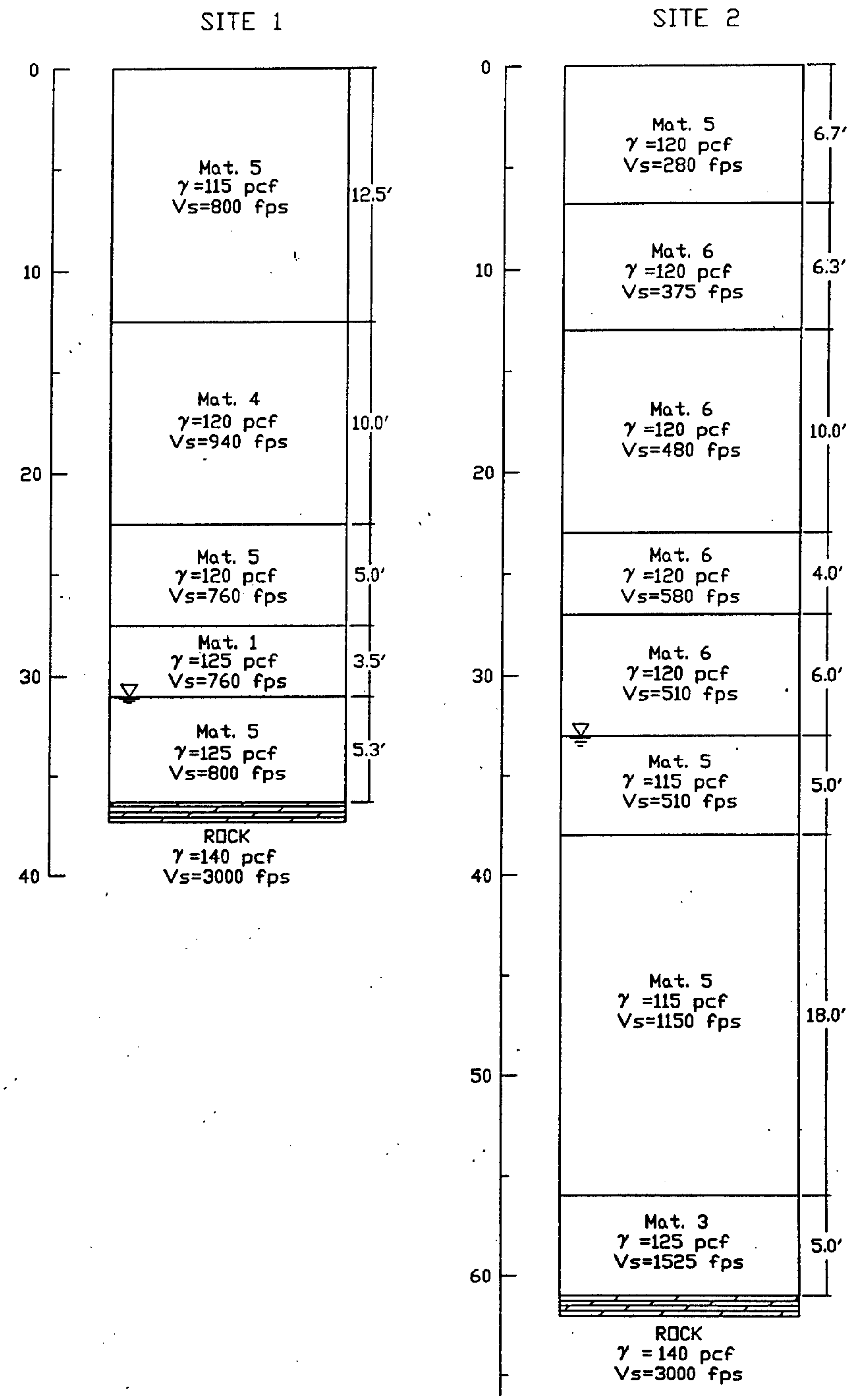


Figure 3.3 Idealization of PORTS sites 1 and 2

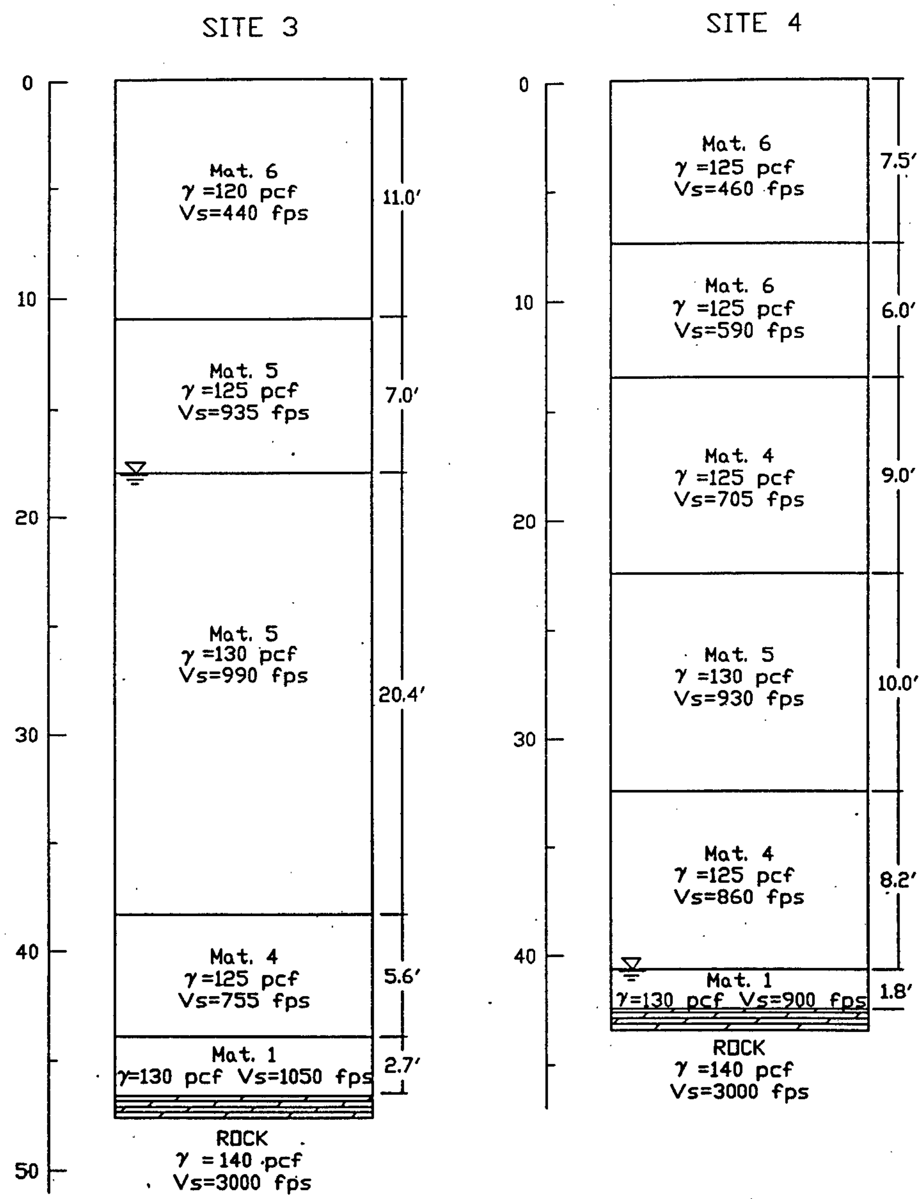


Figure 3.4 Idealization of PORTS sites 3 and 4

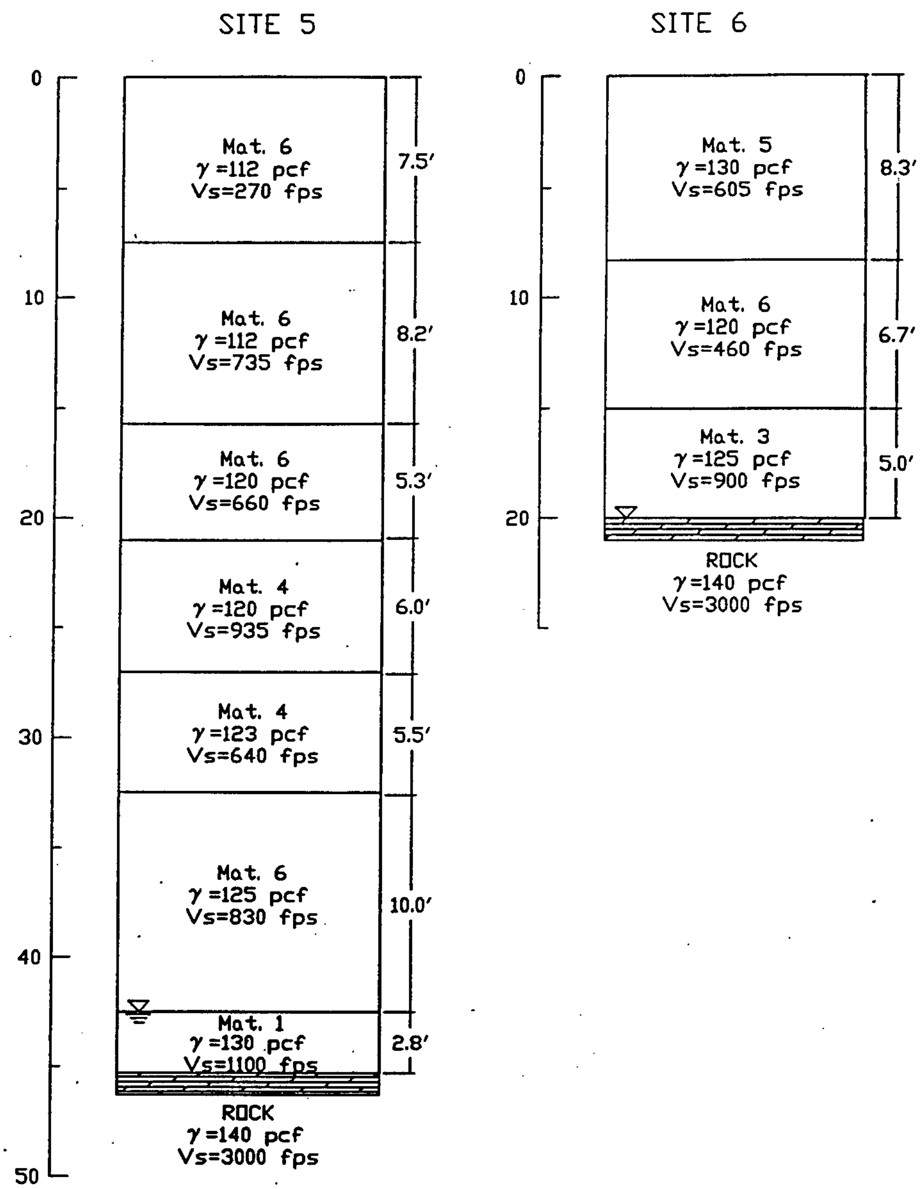


Figure 3.5 Idealization of PORTS sites 5 and 6

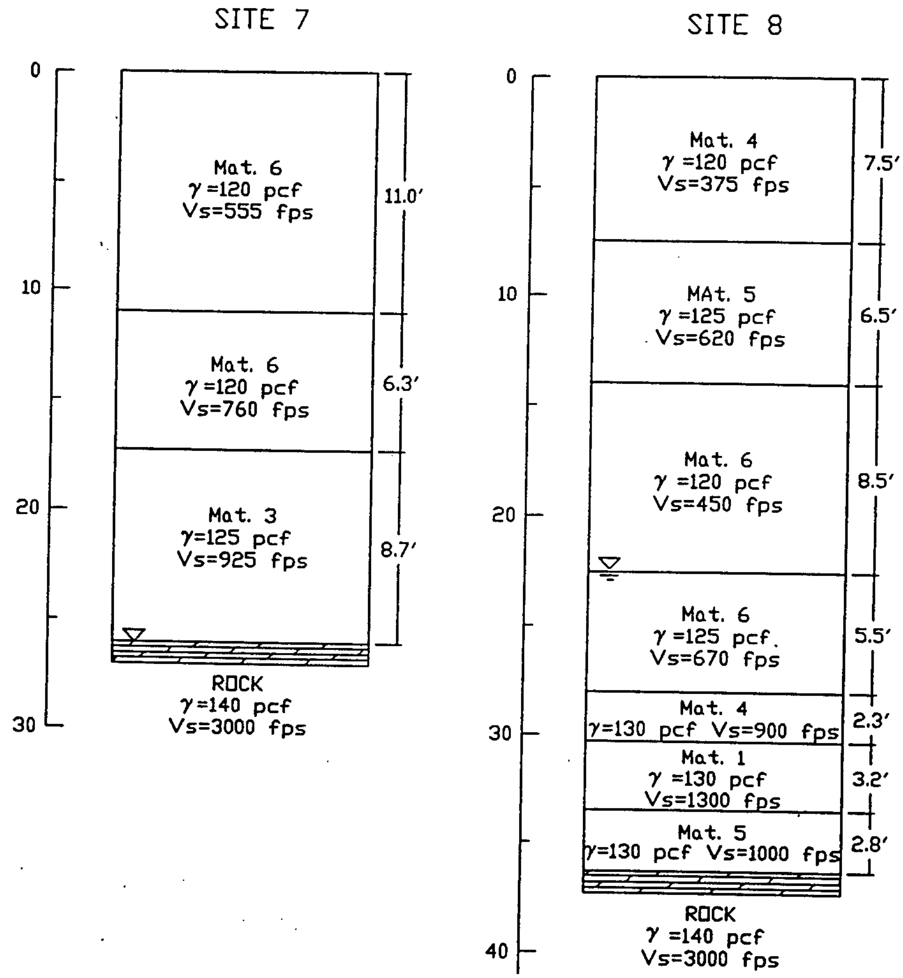


Figure 3.6 Idealization of PORTS sites 7 and 8

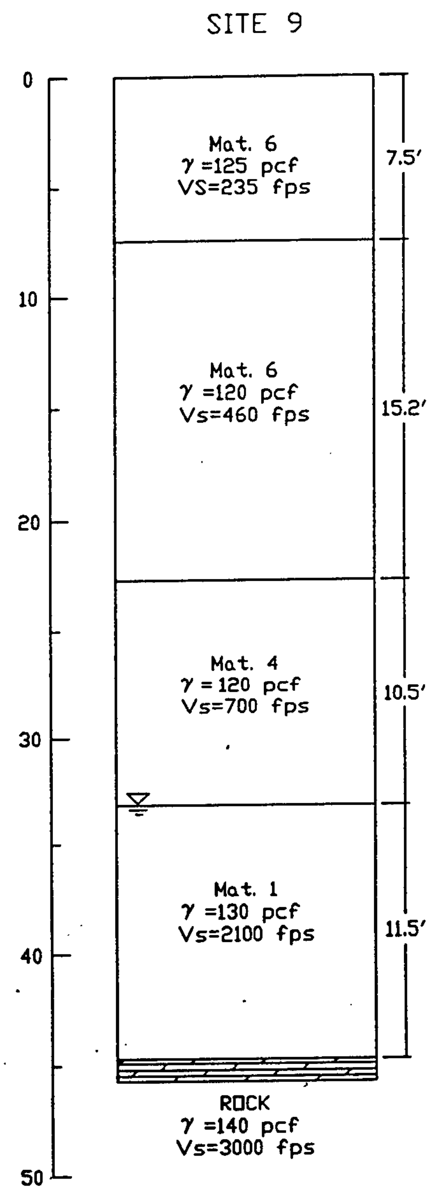


Figure 3.7 Idealization of PORTS site 9

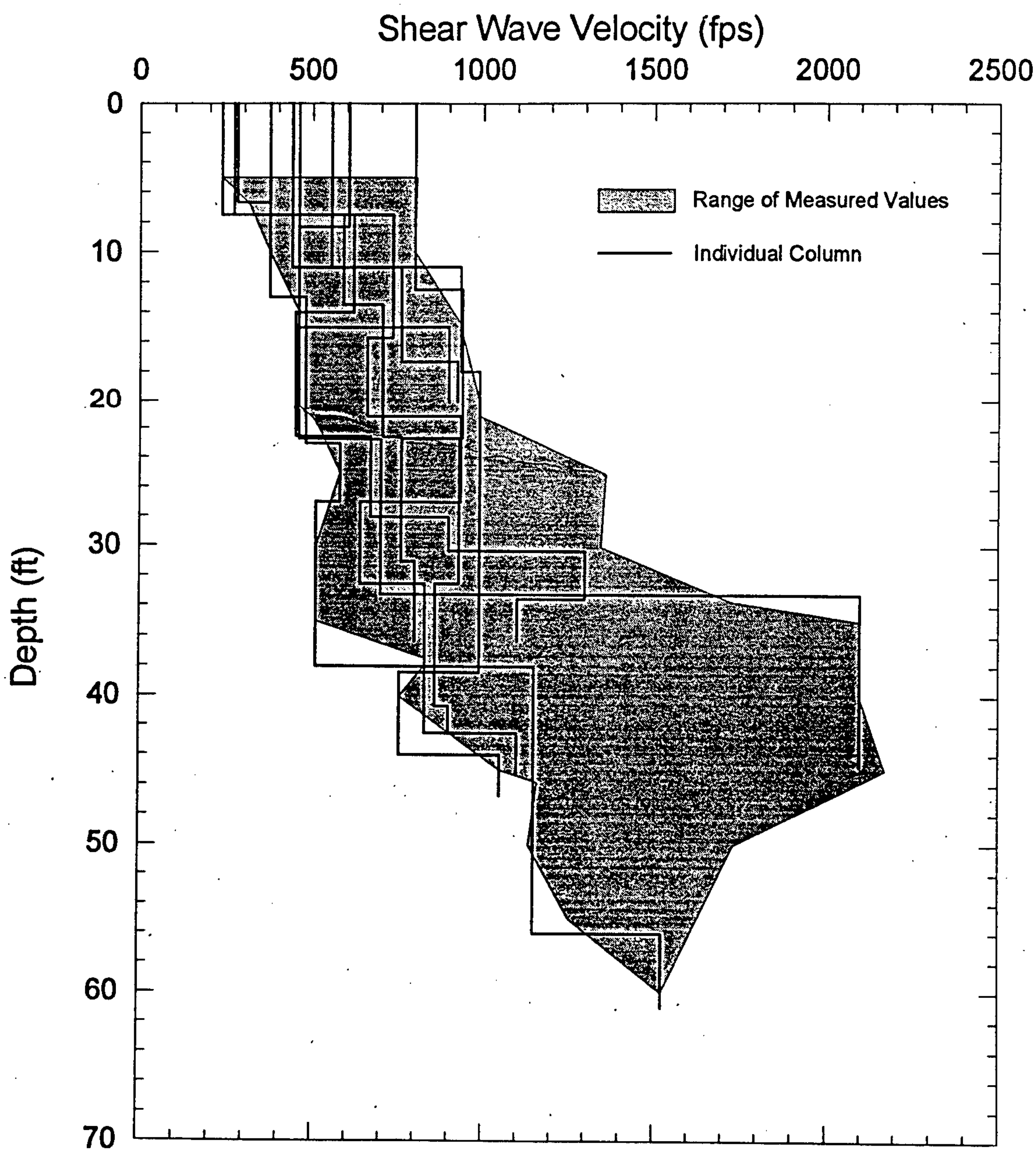


Figure 3.8 Comparison between idealized shear wave velocity profiles and range of measured velocities at PORTS

s1-9.grf

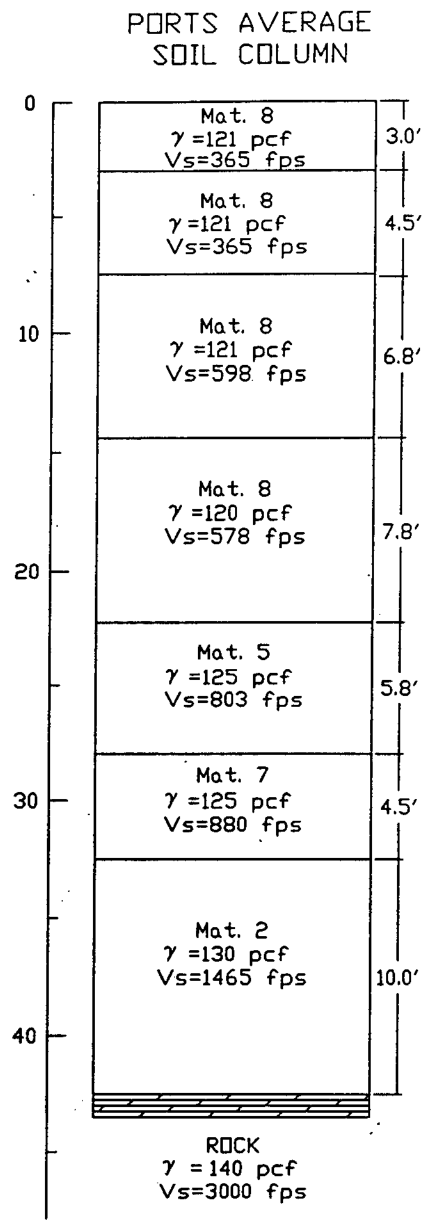


Figure 3.9 Best Estimate column idealization for PORTS

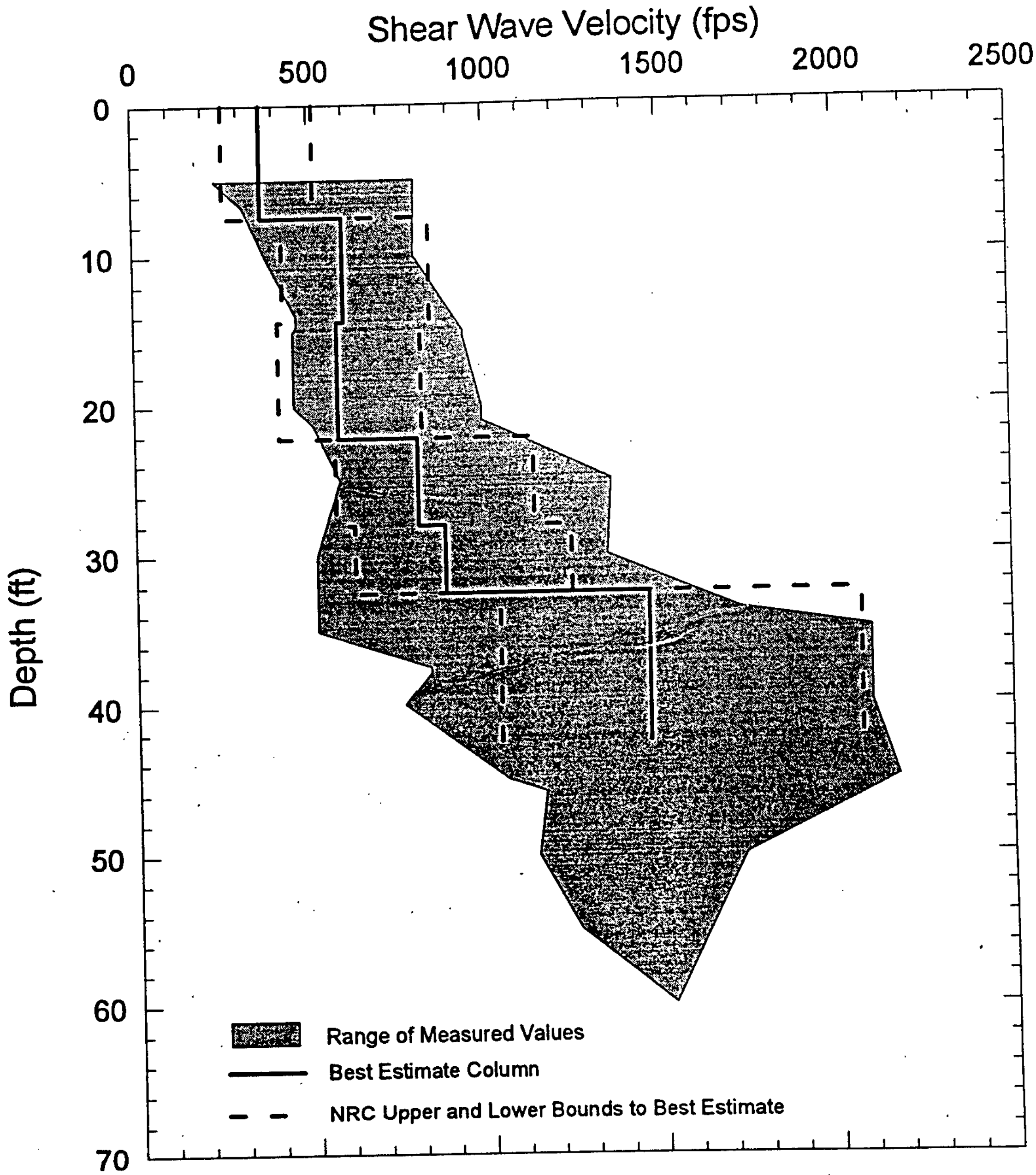
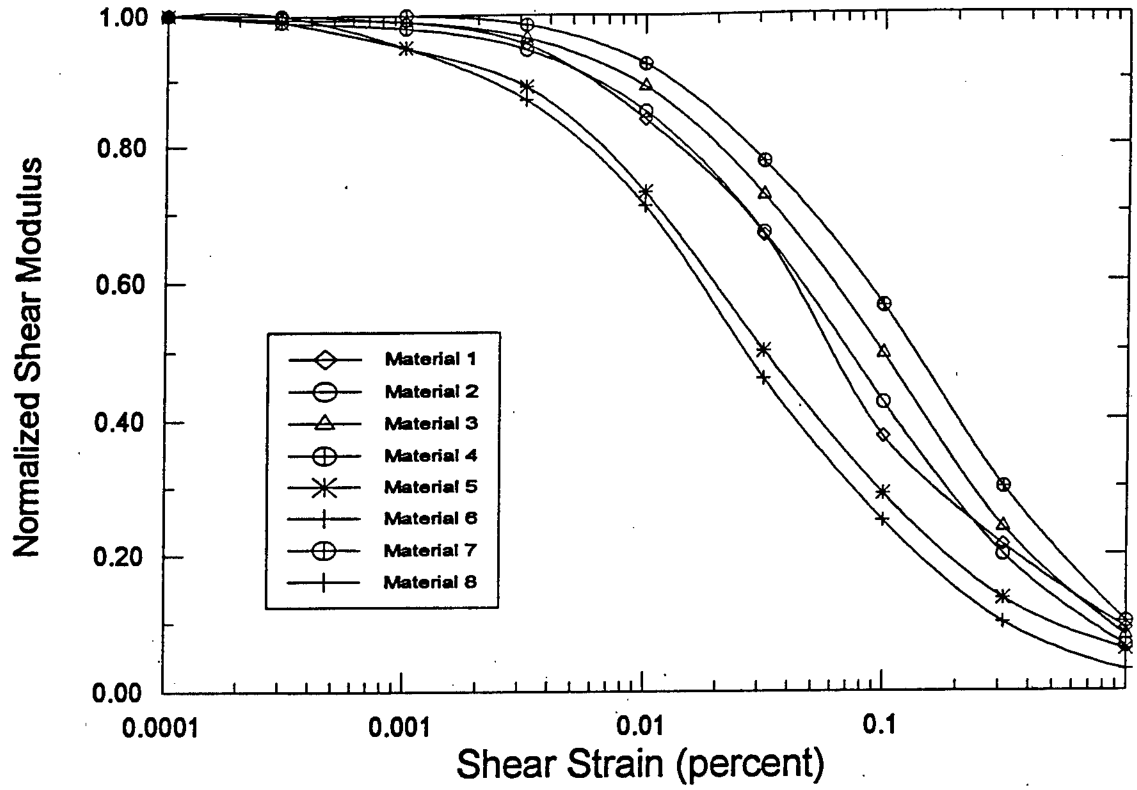


Figure 3.10 Comparison between shear wave velocity profile for best estimate column and range of velocities measured at PORTS

savg.grf



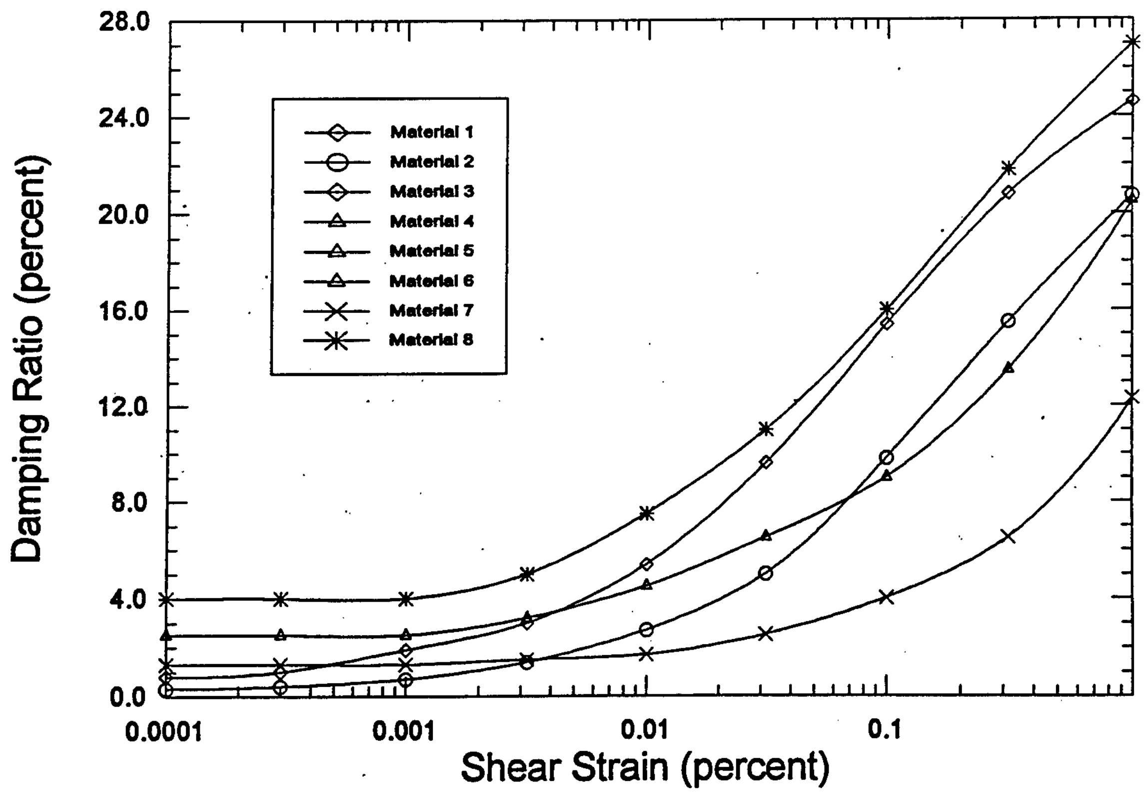


Figure 3.11 Variation in normalized shear modulus and damping ratio with effective shear strain used for shallow site calculations

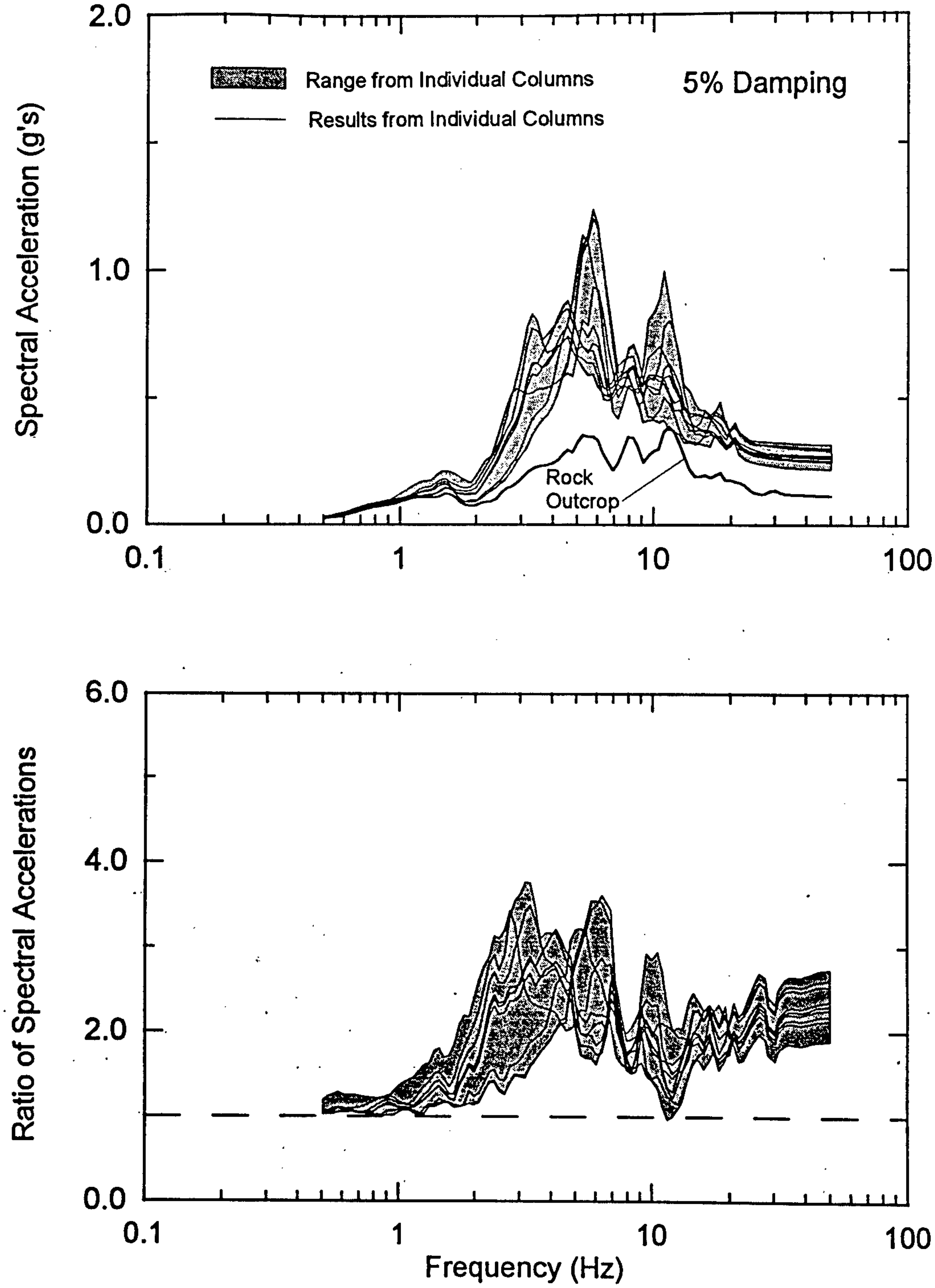


Figure 3.12 Response spectra calculated for individual sites at PORTS and M5 event

wes5a.grf

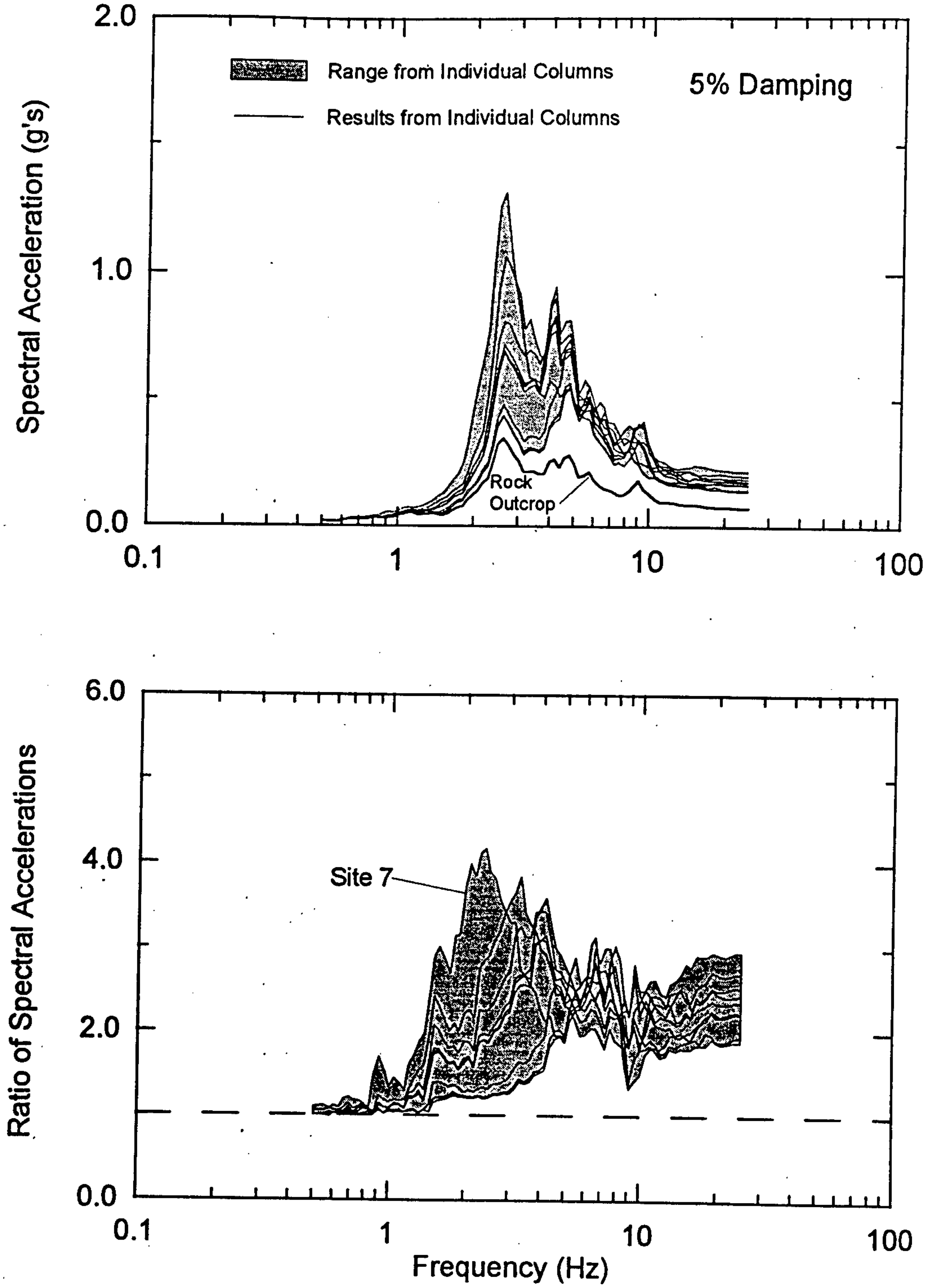


Figure 3.13 Response spectra calculated for individual sites at PORTS and M6 event

wes6a.grf

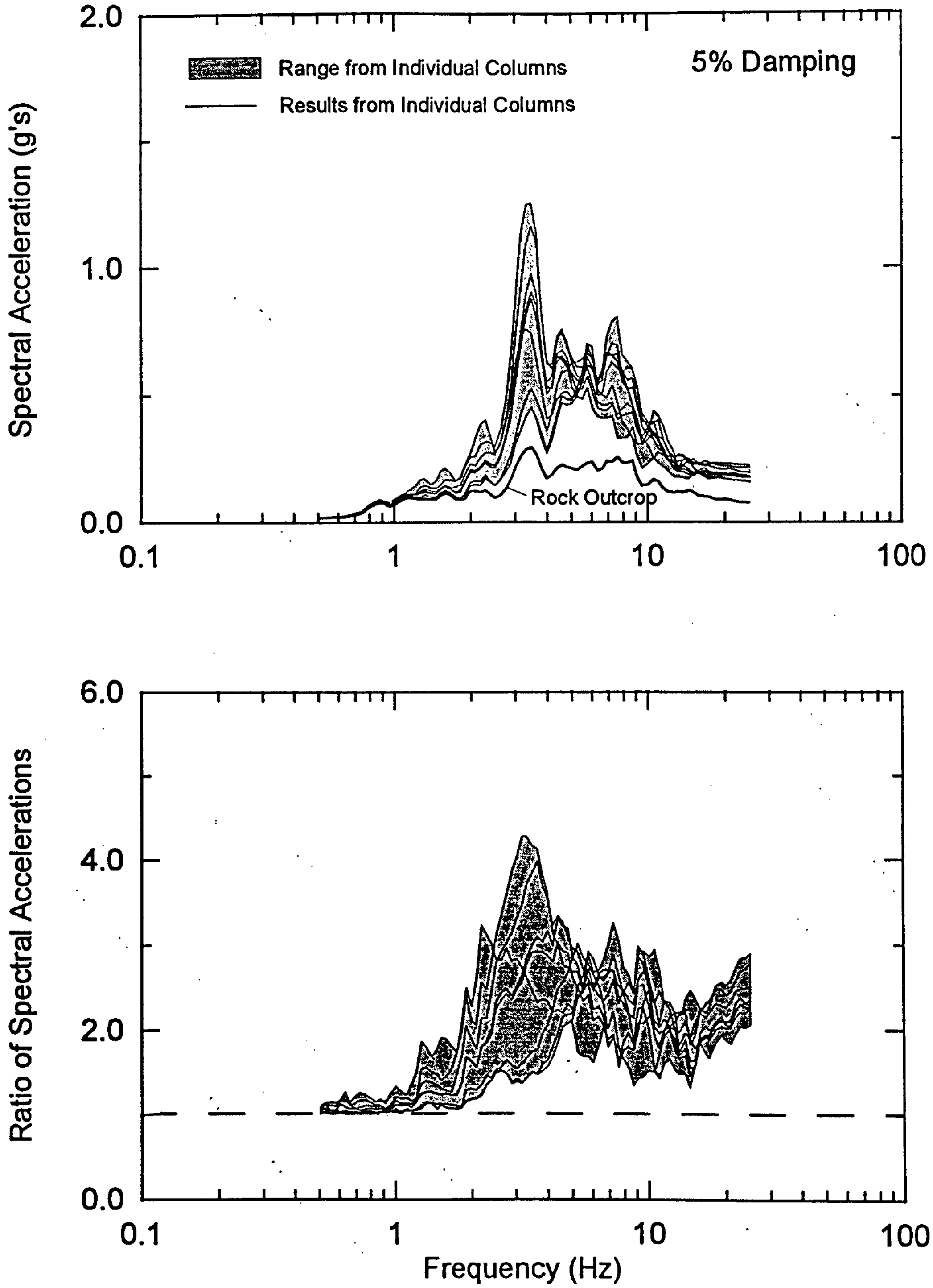


Figure 3.14 Response spectra calculated for individual sites at PORTS and M7 event

wes7a.grf

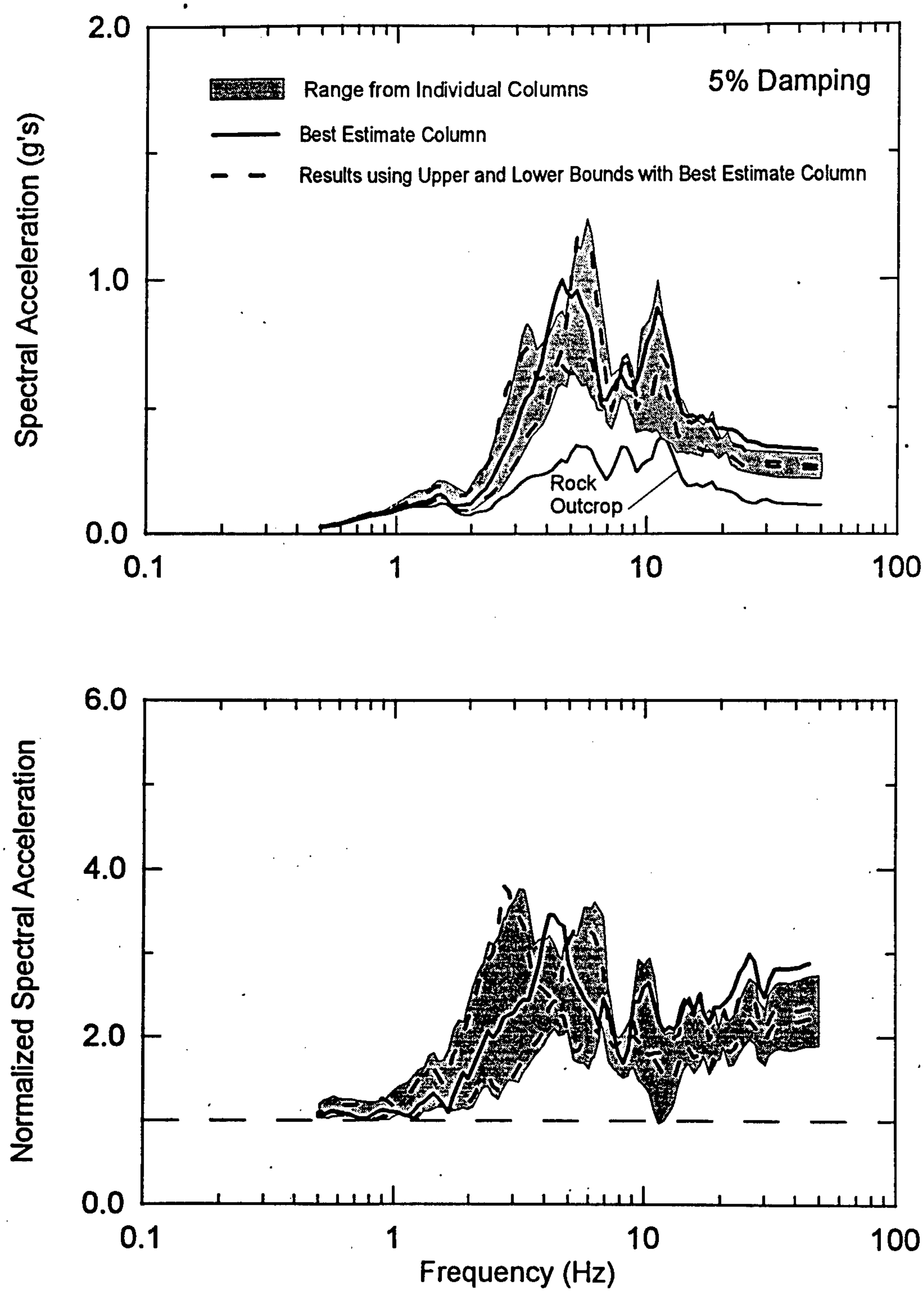


Figure 3.15 Comparison of spectral accelerations for M5 event at PORTS

wesa-cc5.grf

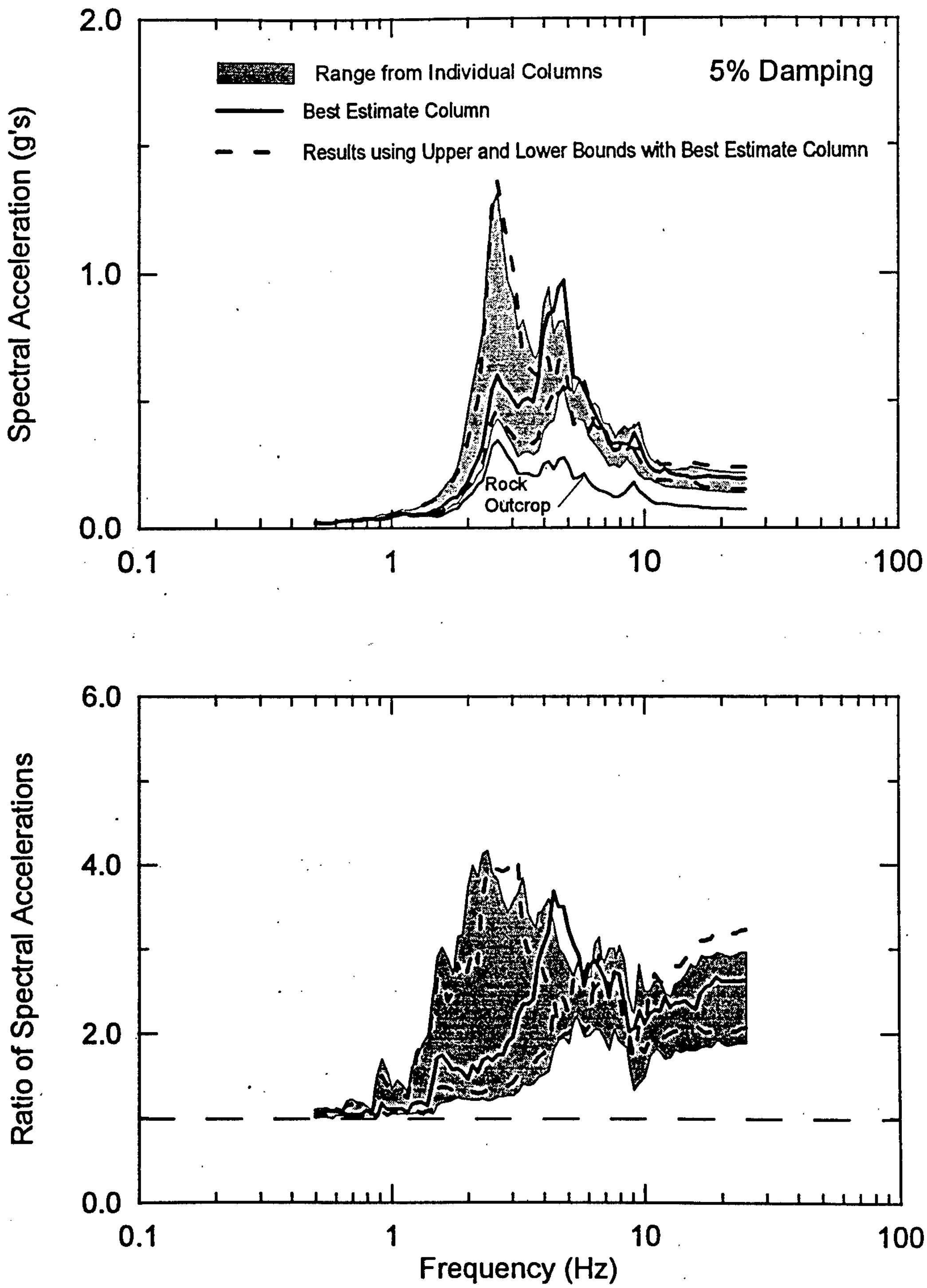


Figure 3.16 Comparison of spectral accelerations for M6 event at PORTS

wesa-cc6.grf

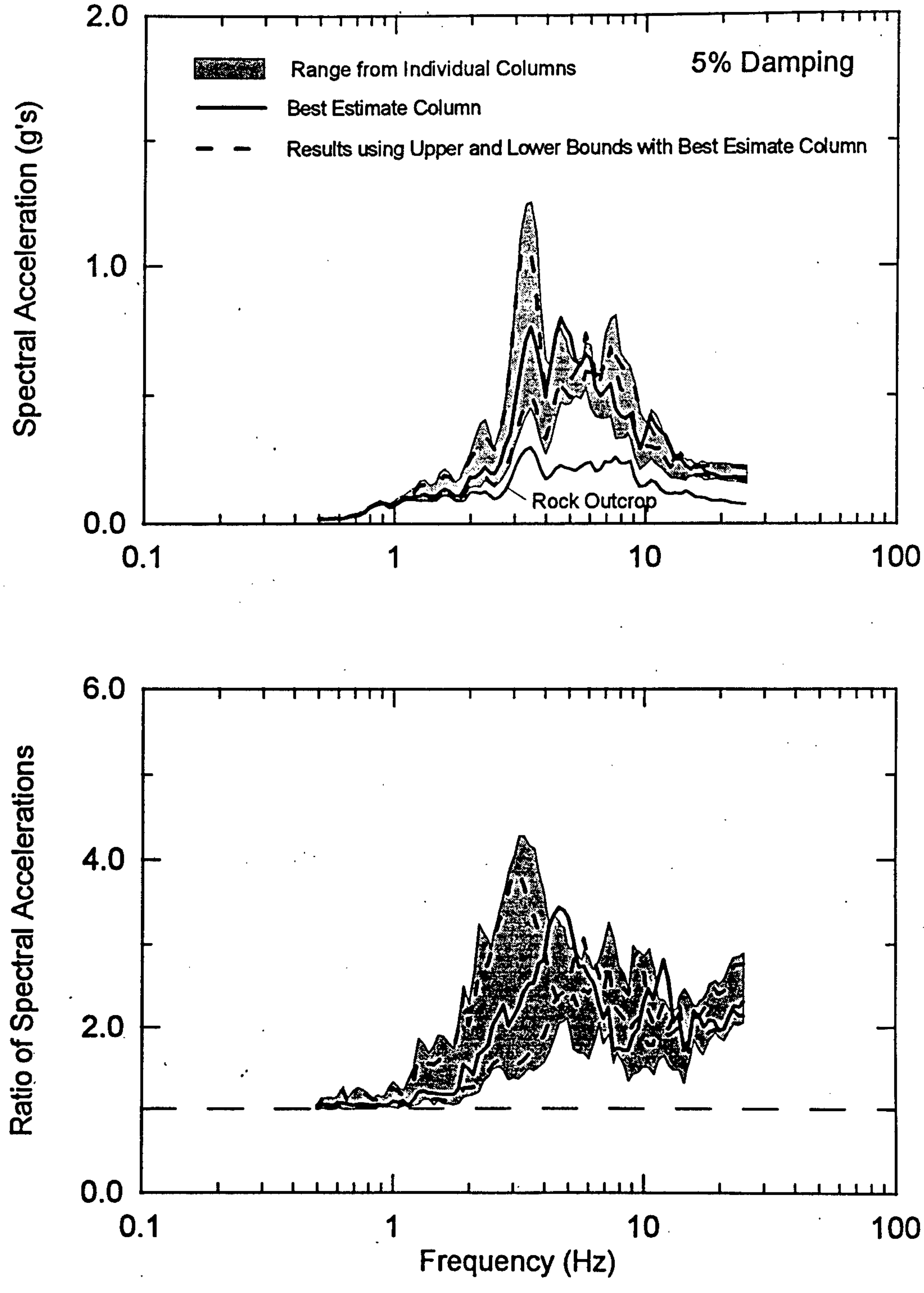


Figure 3.17 Comparison of spectral accelerations for M7 event at PORTS

wese-cc7.grf

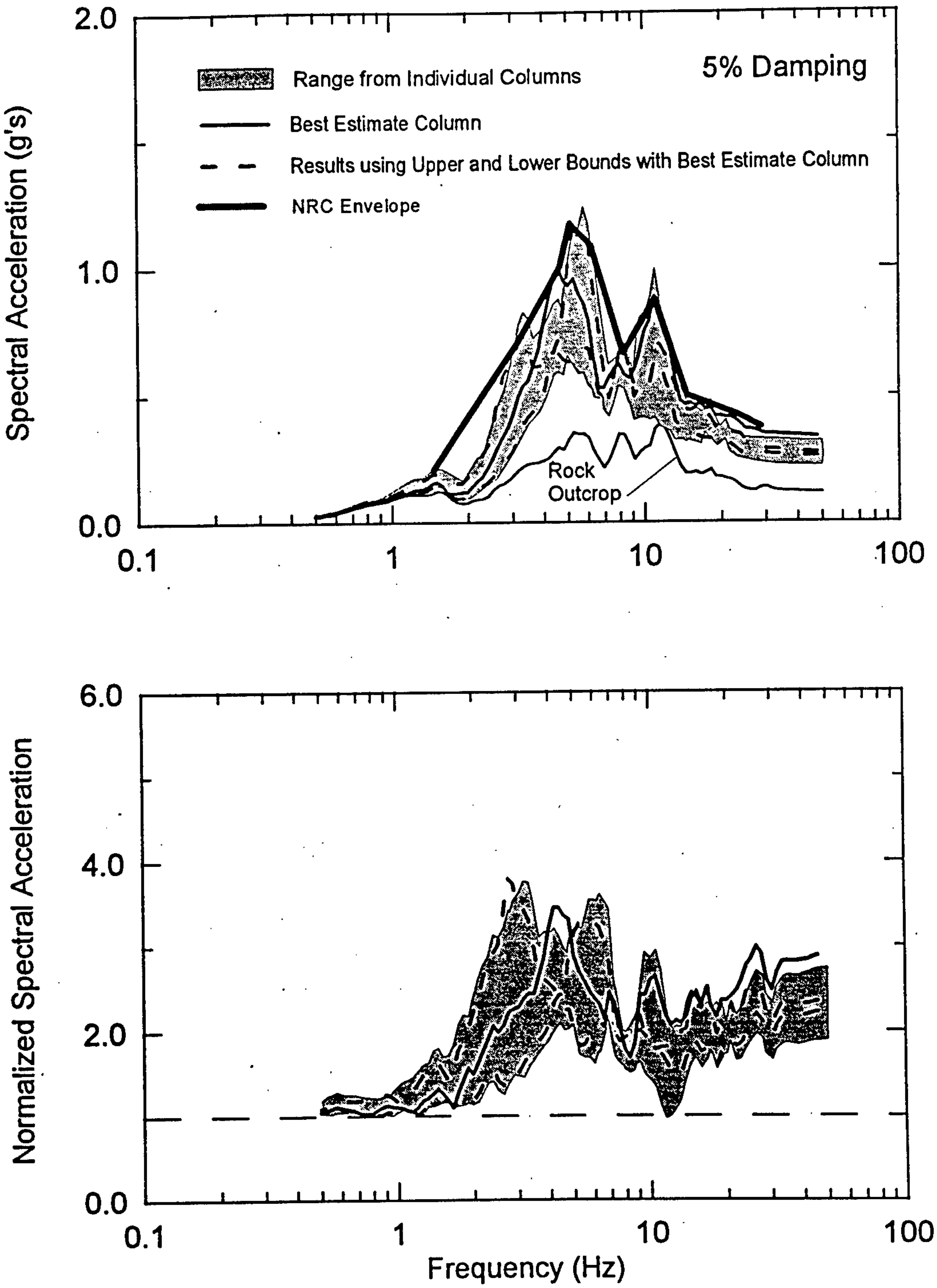


Figure 3.18 Comparison of results for M5 event at PORTS using NRC envelope

wesa-cc5.grf

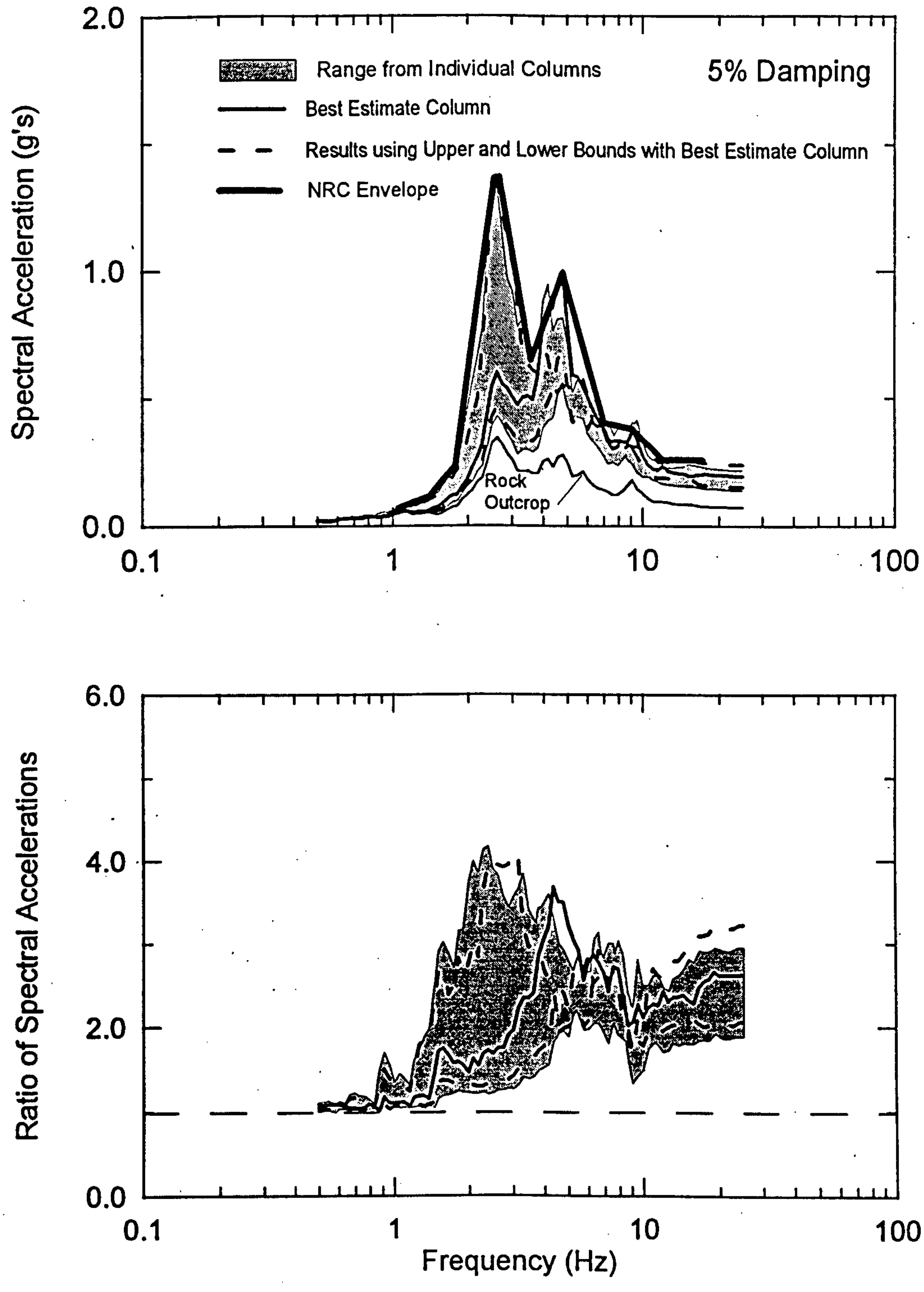


Figure 3.19 Comparison of results for M6 event at PORTS using NRC envelope

wesa-cc6.grf

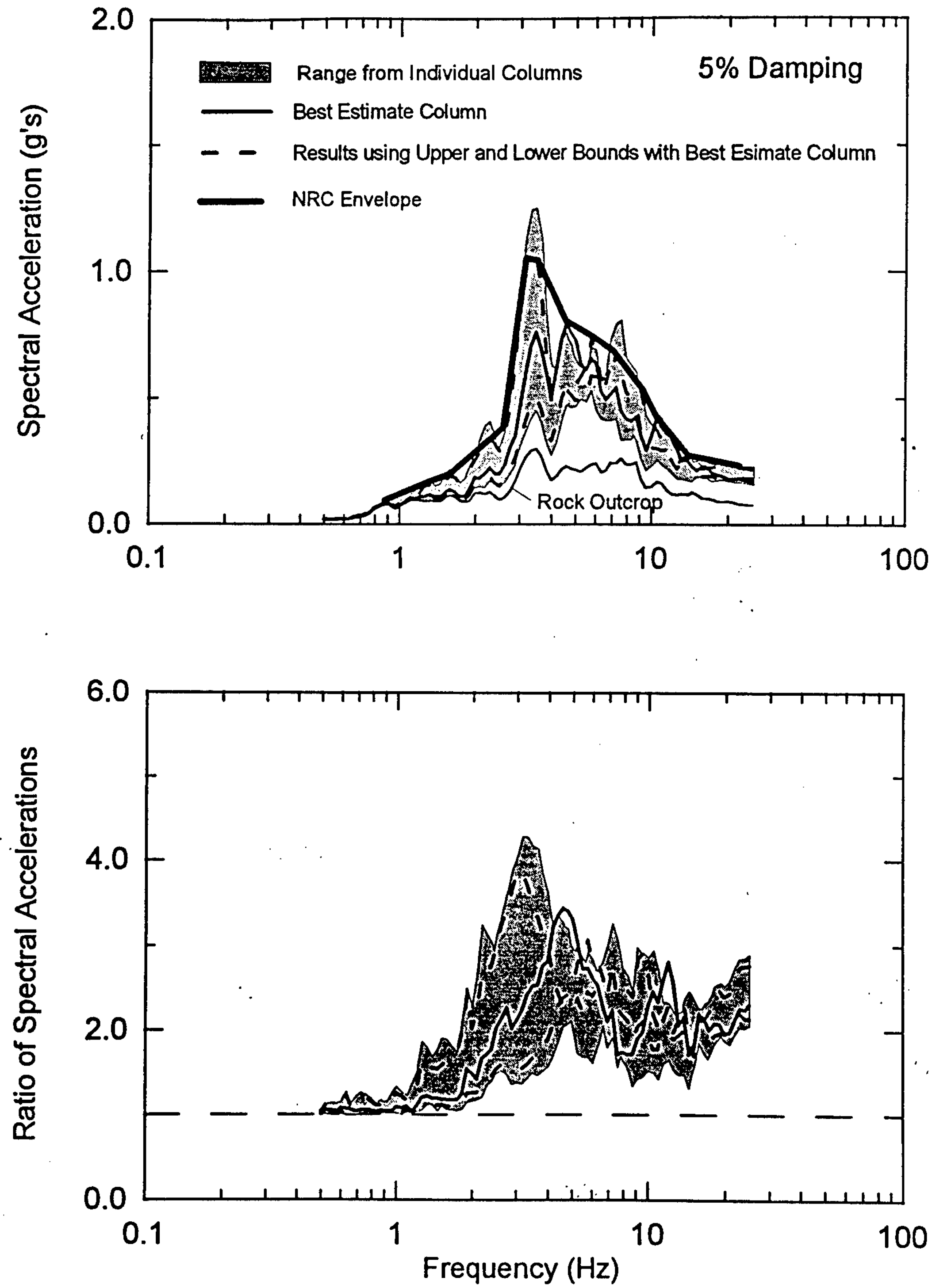
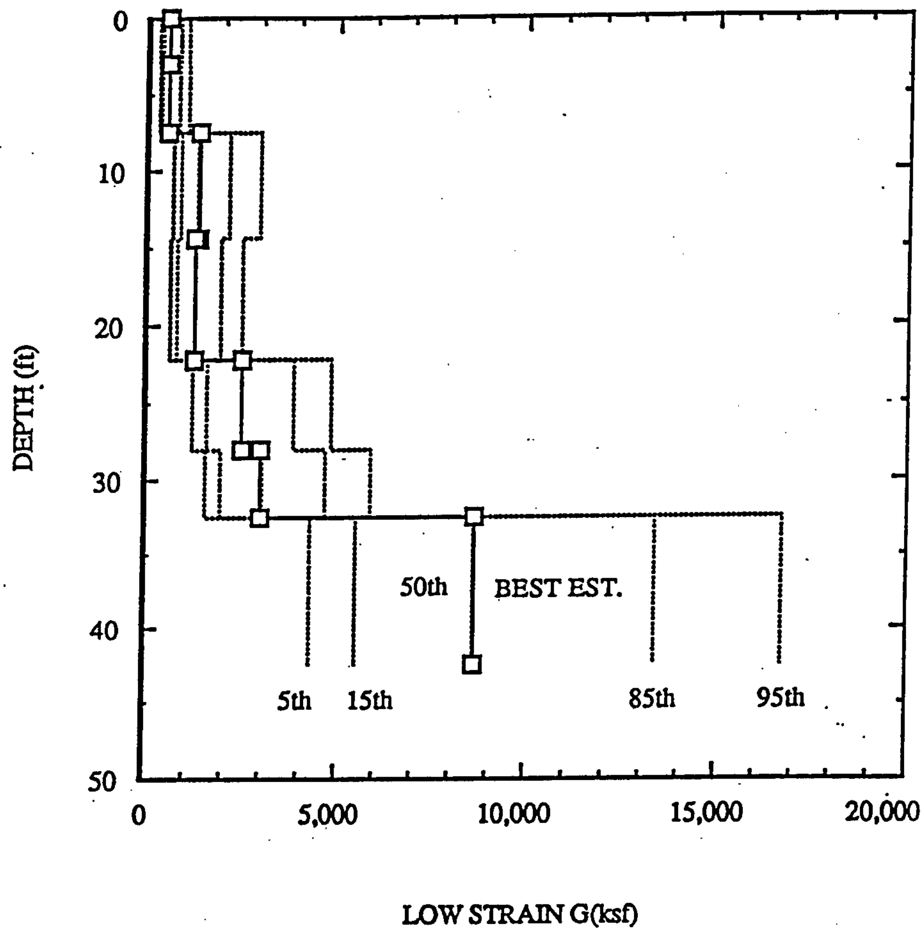


Figure 3.20 Comparison of results for M7 event at PORTS using NRC envelope

wesa-cc7.grf



MAXIMUM SHEAR MODULUS (ksf) AT PORTSMOUTH
USING 1,000 CYCLES
LN-NORMAL DISTRIBUTION FOR SHEAR MODULI
LAYER THICKNESS = BEST ESTIMATE
ROCK OUTCROP MOTION = LOMA PRIETA (M7)

Figure 3.21 Variation of shear modulus corresponding to 5th, 50th, and 95th percentile using Monte Carlo random simulations in shear modulus and M7 event at PORTS

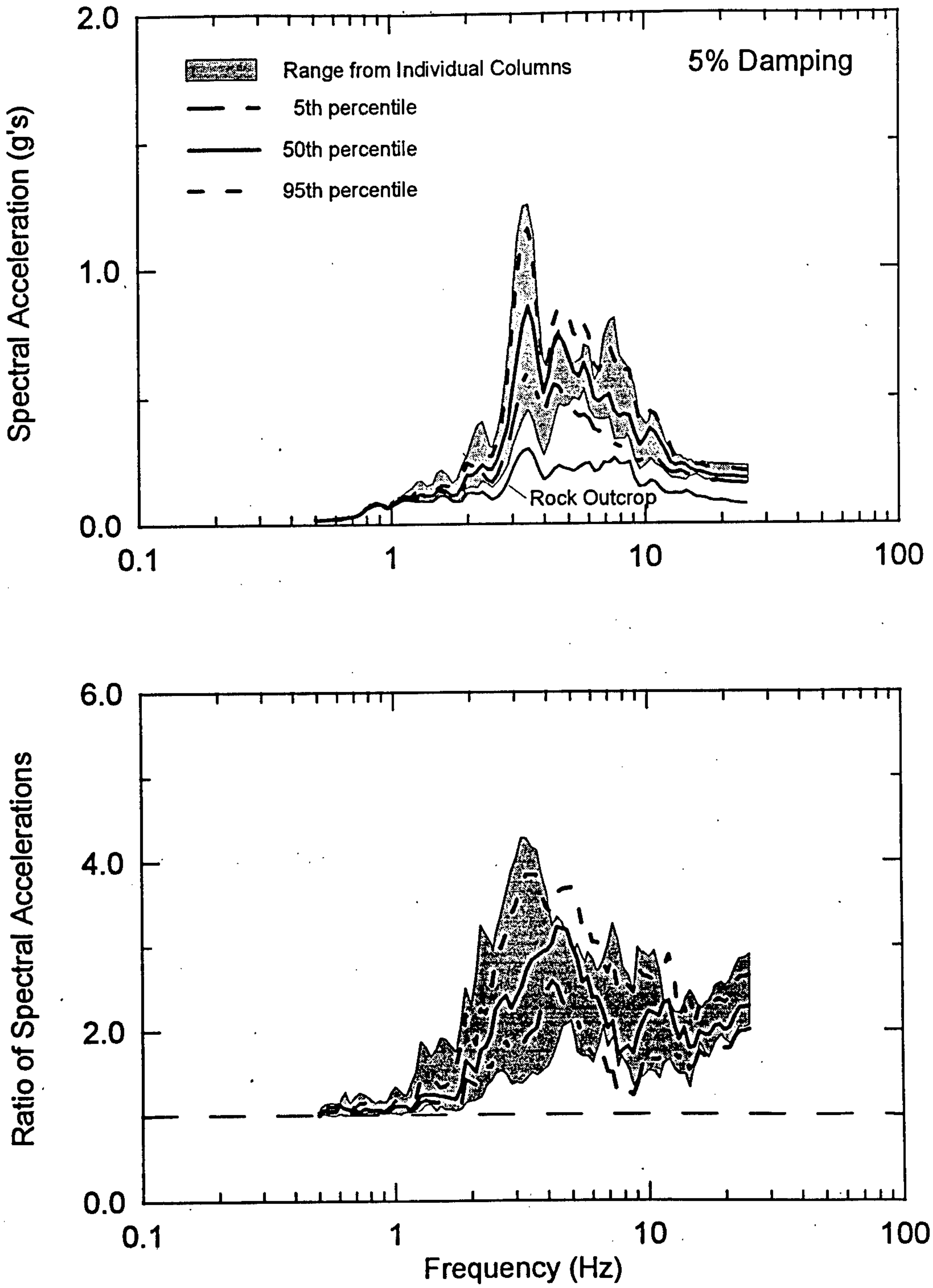


Figure 3.22 Response spectra at 5th, 50th, and 95th percentiles using Monte Carlo random simulations in shear modulus and M7 event at PORTS

wesa-c7a.grf

4 Deep Site Evaluation

An evaluation of the two idealization methods was made using a deep soil site at the DOE Savannah River Site (SRS), South Carolina. The deep soil site is expected to produce a site response with a predominance of low-frequency motions. Any reasonable variation of total soil thickness would be expected to produce a negligible effect on the variation of site response. One synthetic earthquake motion corresponding to a magnitude 7.5 EUS event at a distance of 145 km from the source was applied with peak horizontal acceleration of 0.05 g.

Site Description

The SRS is located about 25 miles southeast of Augusta, Georgia, and is bordered on the west by the Savannah River. The ITP plant is located in Area H of SRS and is generally 40 ft above surrounding grade by means of a controlled embankment fill. A site map of the area, covering approximately 15 acres, is shown in Figure 4.1. This site consists of waste storage tanks and a treatment and separation facility for radioactive wastes.

Site Characterization

The SRS is located on the upper Atlantic Coastal Plain on relatively unconsolidated coastal plain sediments eroded from crystalline igneous and metamorphic rocks. The Paleozoic crystalline rock basement exists at about a depth range of 800 to 1,200 ft and is covered by late cretaceous and then tertiary sediments. The depth to the basement at this site was around 970 ft.

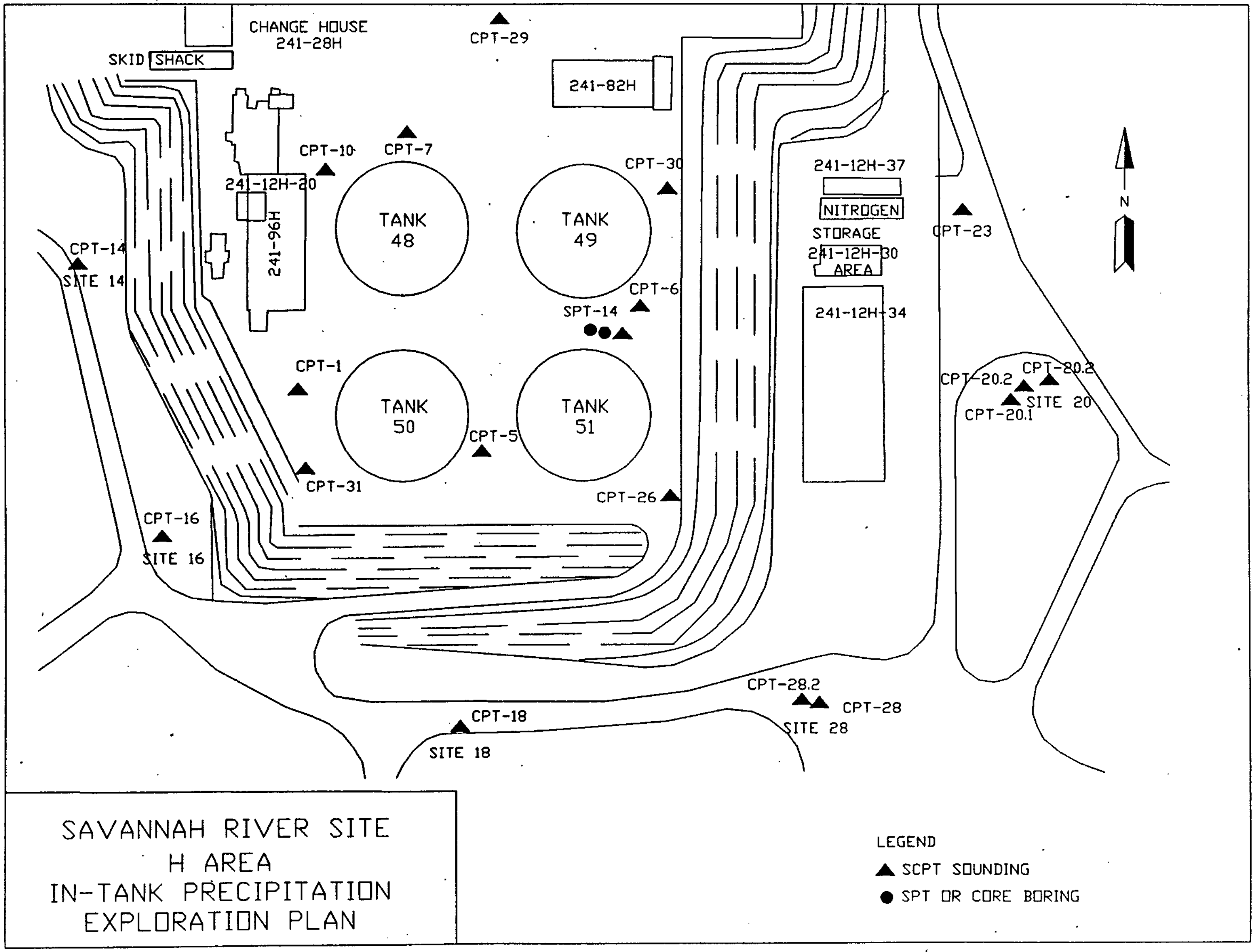


Figure 4.1 **Site Plan of In-Tank Precipitation (ITP) Project at SRS showing locations of sites used for earthquake response evaluations**

Site Idealization

As seen in Figure 4.1, several clusters of subsurface investigations, including seismic cone penetrometer tests (SCPT), standard penetration test (SPT), dilatometer test (DMT), and undisturbed sampling, exist in the vicinity of the ITP tanks. These data were typically available to depths of investigation of about 150 ft. The idealization of deeper deposits was taken from a single deep borehole investigation.

Data from four of these clusters located off the fill area (clusters 14, 18, 20, and 28) were used to develop individual soil columns that did not include the fill. The closest spacing between these individual sites is about 250 ft; the farthest distance between two sites is about 650 ft.

The characterization of subsoils used to derive the four idealized individual columns are shown in Figures 4.2 through 4.5. The soil layering was determined using the SCPT. Unit weights were obtained from data obtained from numerous undisturbed specimens and averaged for each layer. Shear wave velocities were measured using the SCPT. Seventeen profiles of measured shear wave velocity over the 15-acre area are shown in Figure 4.6. The shear wave velocities measured in natural soils (below elevation 290) are generally between 650 and 1,700 fps, with a general average of about 1,100 fps that does not seem to increase much with depth over the upper 200 ft of the soil column .

The idealized profiles of shear wave velocity at depths less than about 150 ft for the four individual columns (refer to Figures 4.2 through 4.5) are compared with all measured values in Figure 4.7. The four profiles generally fall well within the range of measure values, defining the upper or lower bounds at only a few depths.

The variation of shear wave velocity for the upper 170 ft of the idealized best-estimate column is shown in Figure 4.8. The profile of shear wave velocity for the best-estimate column compared with all measured values are shown in Figure 4.9. This profile does not vary considerably which is likely caused by "averaging"

numerous values at each depth without associating high and low values with particular soil layers. Also shown are the upper and lower bounds as defined in Chapter 2. These bounds provide a reasonable representation of the range of measured values.

The measured and idealized shear wave velocity profiles for greater depths are compared in Figure 4.10. The data for depths greater than 200 ft were obtained from a single deep boring located at another site at the SRS. The idealized profile used for individual and best-estimate columns differ slightly to better represent the data and to accommodate the 20-layer restriction on input specification for *SHAKE*. The shear wave velocity for bedrock was 11,500 fps. All column heights were adjusted to have about the same height of 970 ft by adjusting the thickness of the layer with the bottom at a depth of 165 ft (V_s=1,400 fps).

The variation of normalized shear modulus and damping ratio with effective shear strain used for the deep site evaluation are shown in Figure 4.11. Materials 1 through 3 were used on individual columns for layers within the upper 150 ft. The selection among these curves was made based on the plasticity index (Vucetic and Dobry 1981). Material 4 was used in the best-estimate column for all layers above 150 ft. Materials 5 and 6 were used at depths below 150 ft and were consistent for the individual columns and the best-estimate column. In general, the effective shear strains for the final iterated solution ranged from 0.0006 to 0.05 percent.

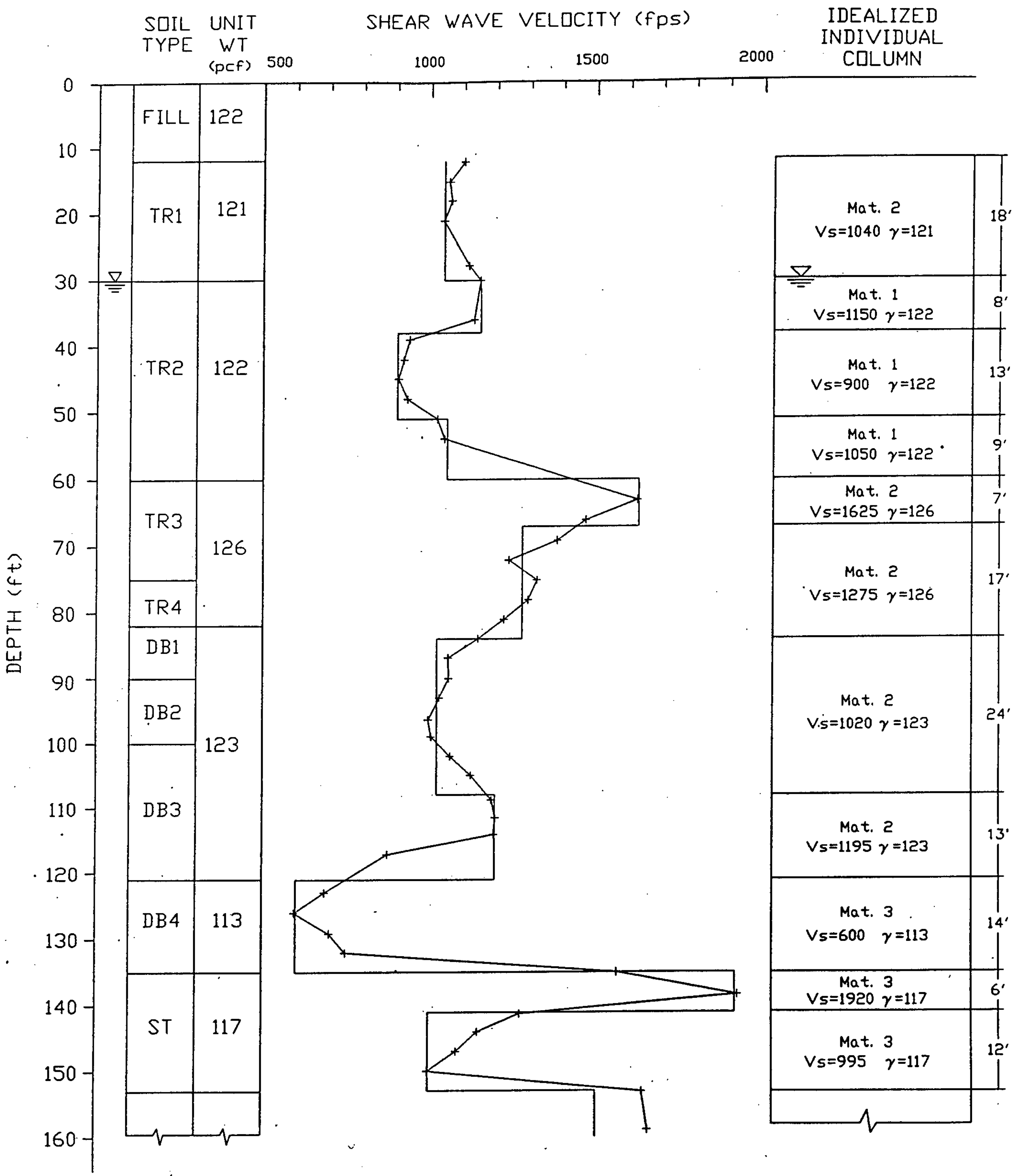


Figure 4.2 Characterization and idealization of ITP site 14 at SRS

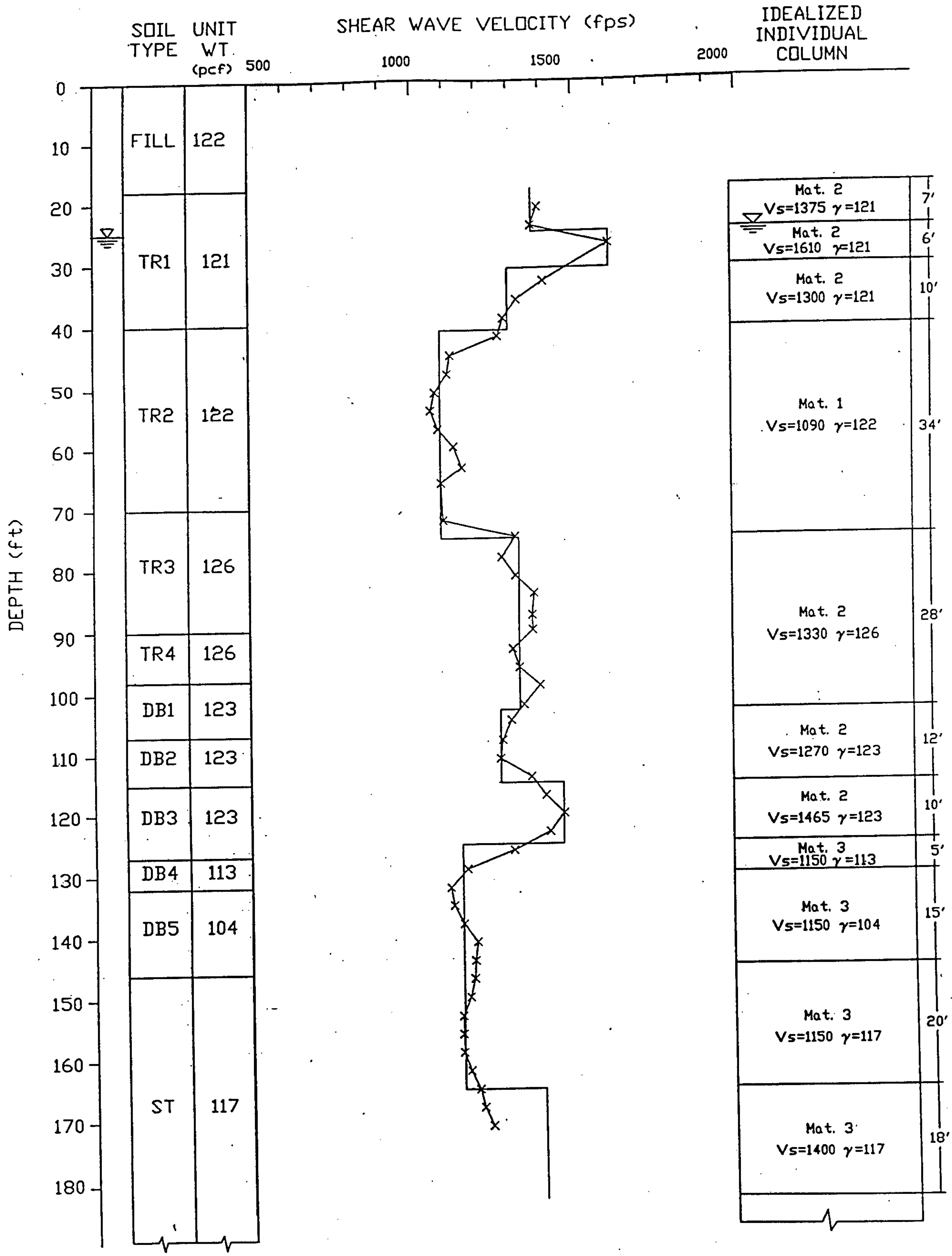


Figure 4.3 Characterization and idealization of ITP site 18 at SRS

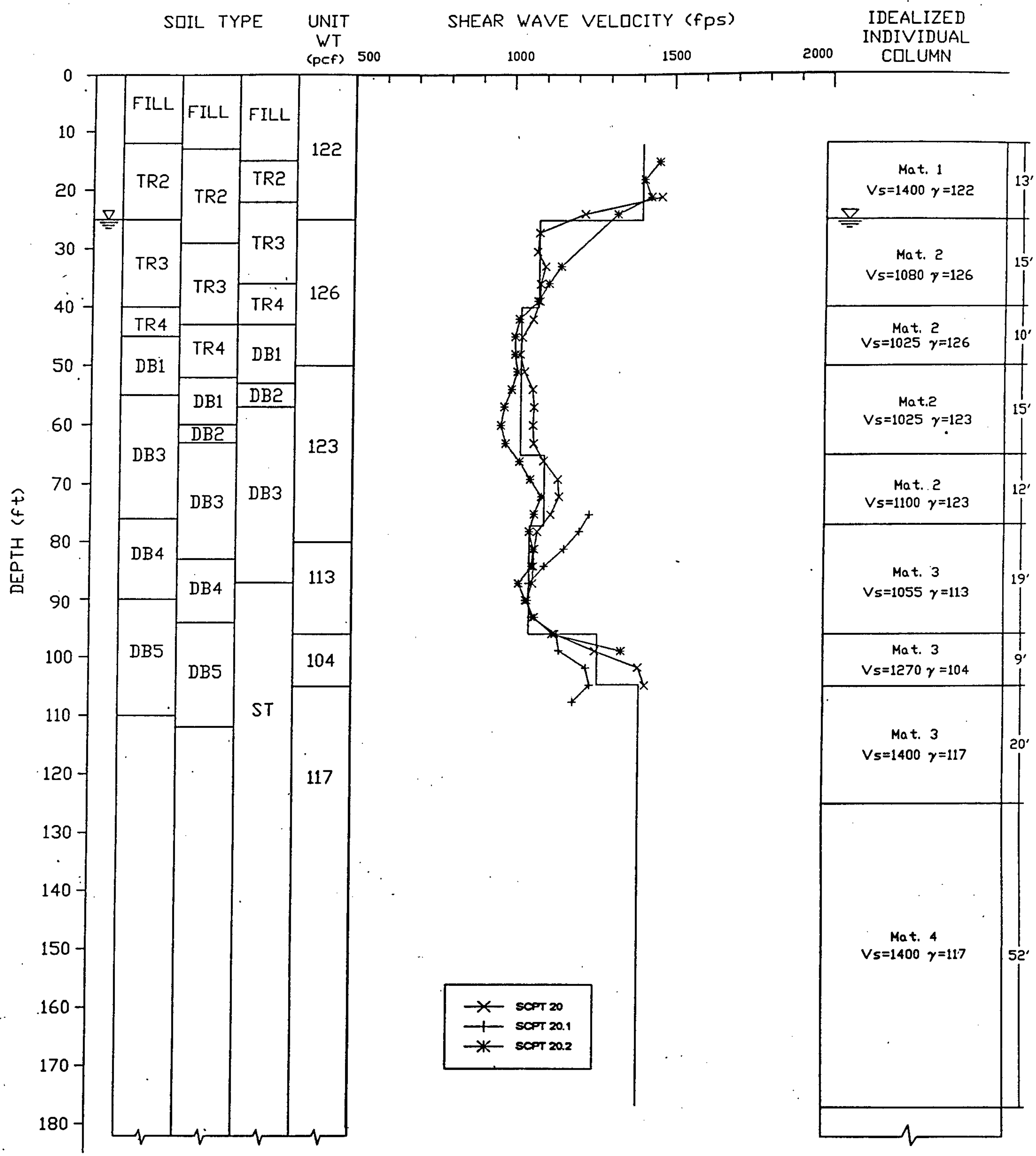


Figure 4.4 Characterization and idealization of ITP site 20 at SRS

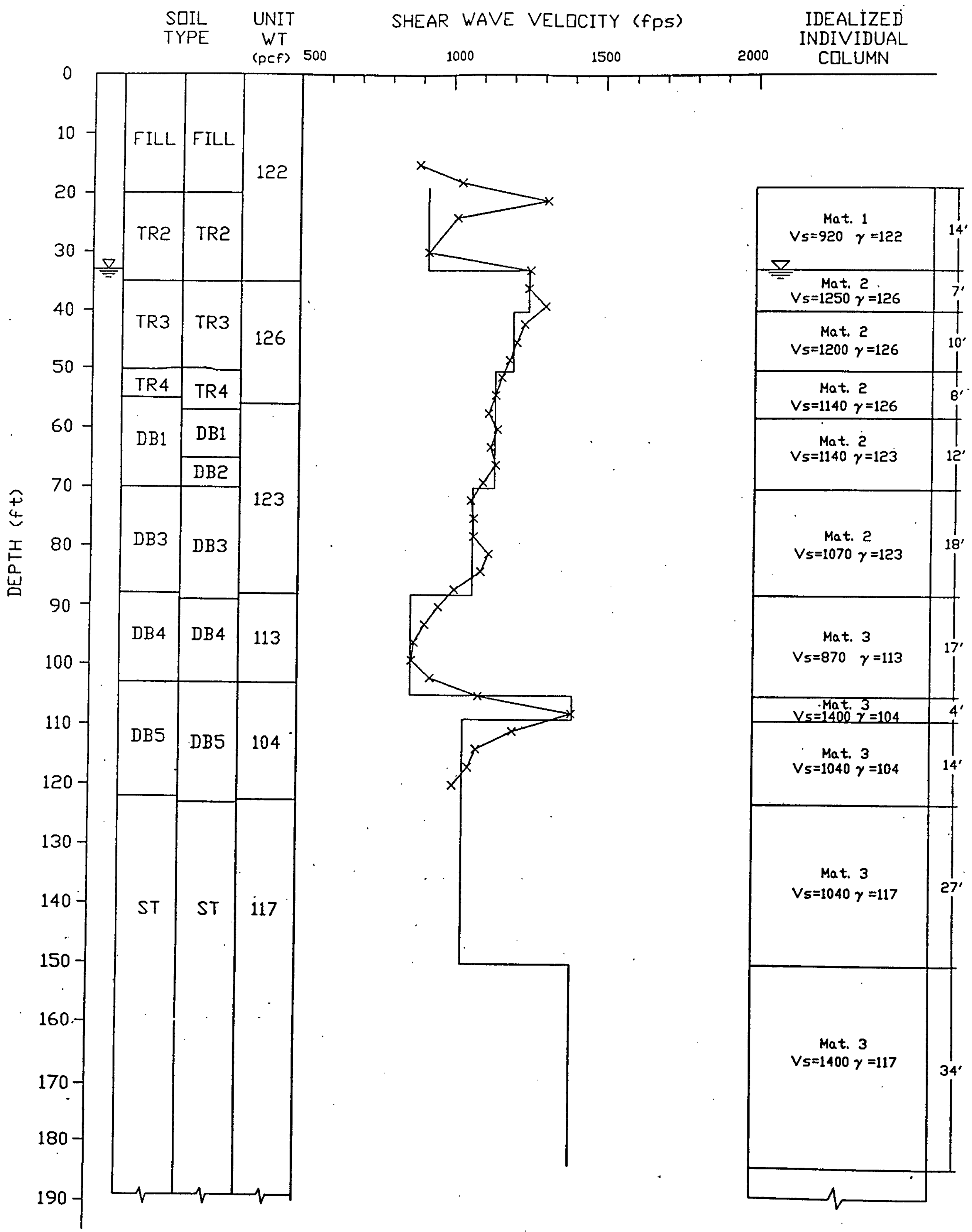


Figure 4.5 Characterization and idealization of ITP site 28 at SRS

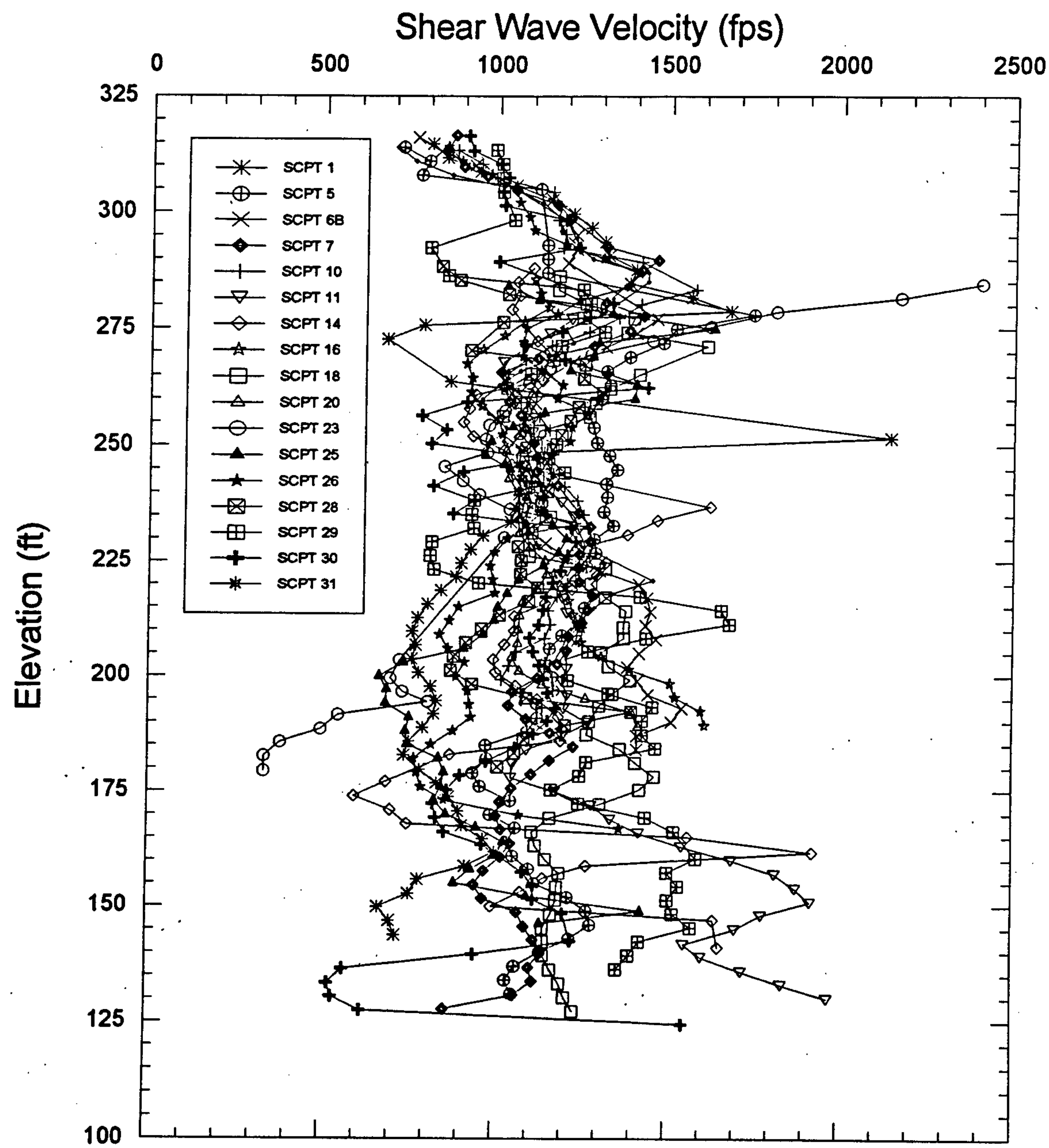


Figure 4.6 Variation in shear wave velocity in upper 150 feet for all ITP sites at SRS

all_elev.grf

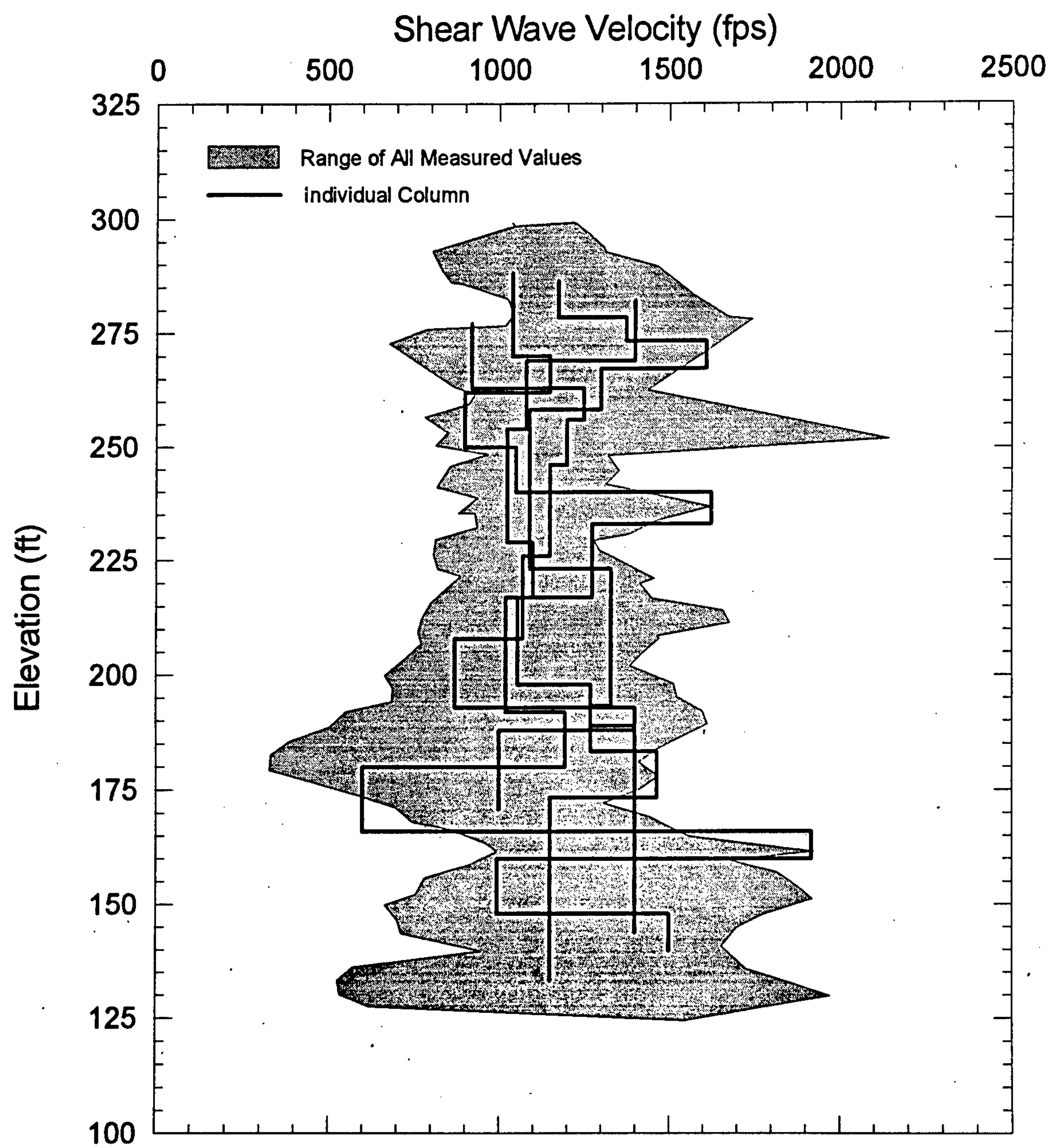


Figure 4.7 Comparison between idealized shear wave velocity profiles in upper 150 feet and range of measured velocities at ITP of SRS

vs_mg.grf

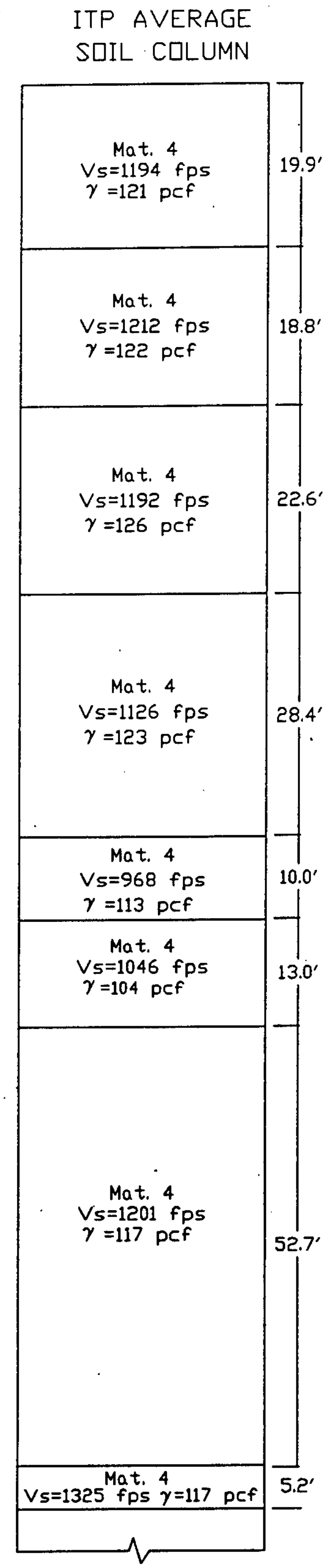


Figure 4.8 Best estimate column idealization for ITP at SRS

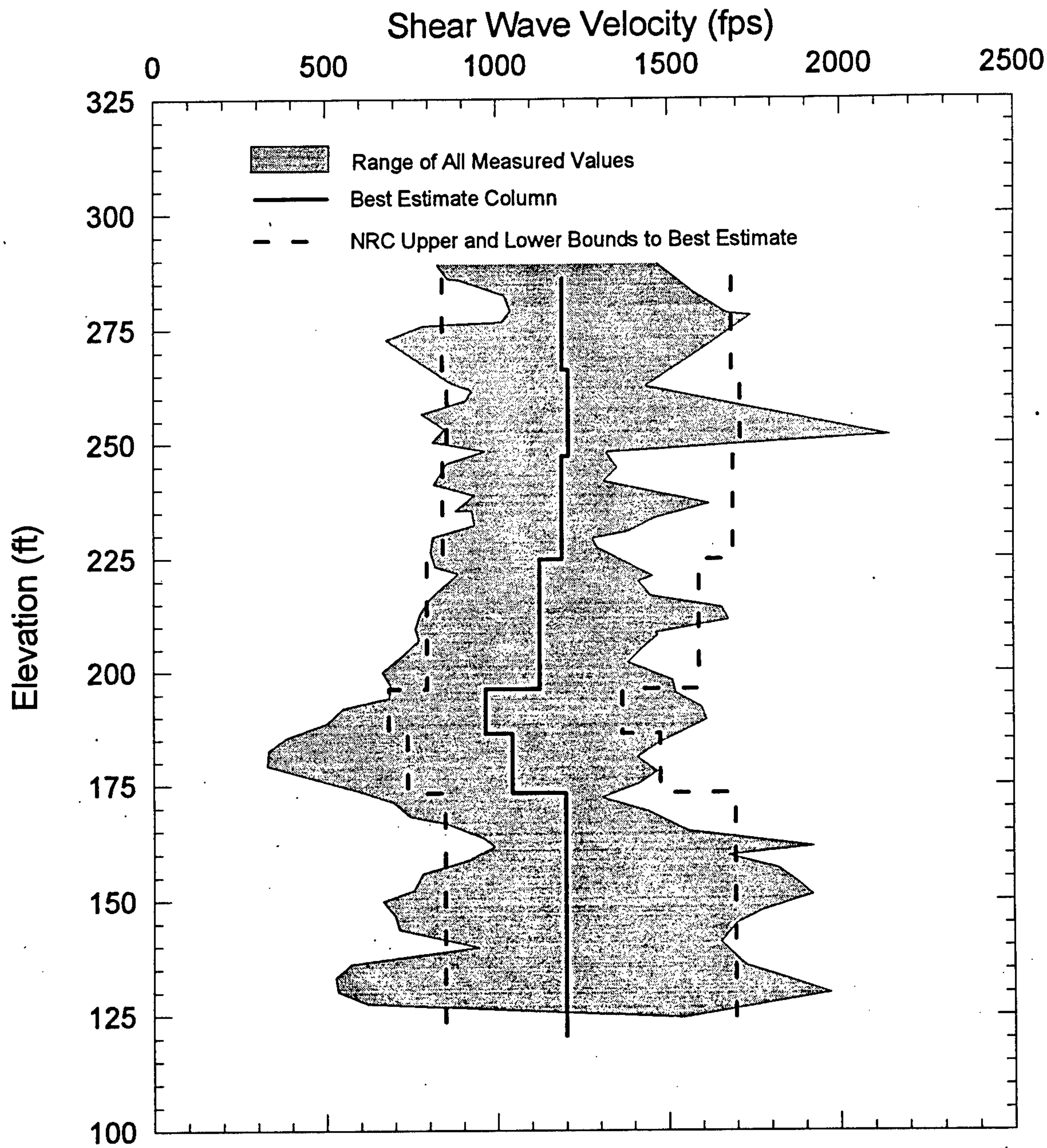


Figure 4.9 Comparison between shear wave velocity profile for best estimate column, including upper and lower bounds in shear wave velocities, and measured range of velocities at ITP of SRS

vs_cc.grf

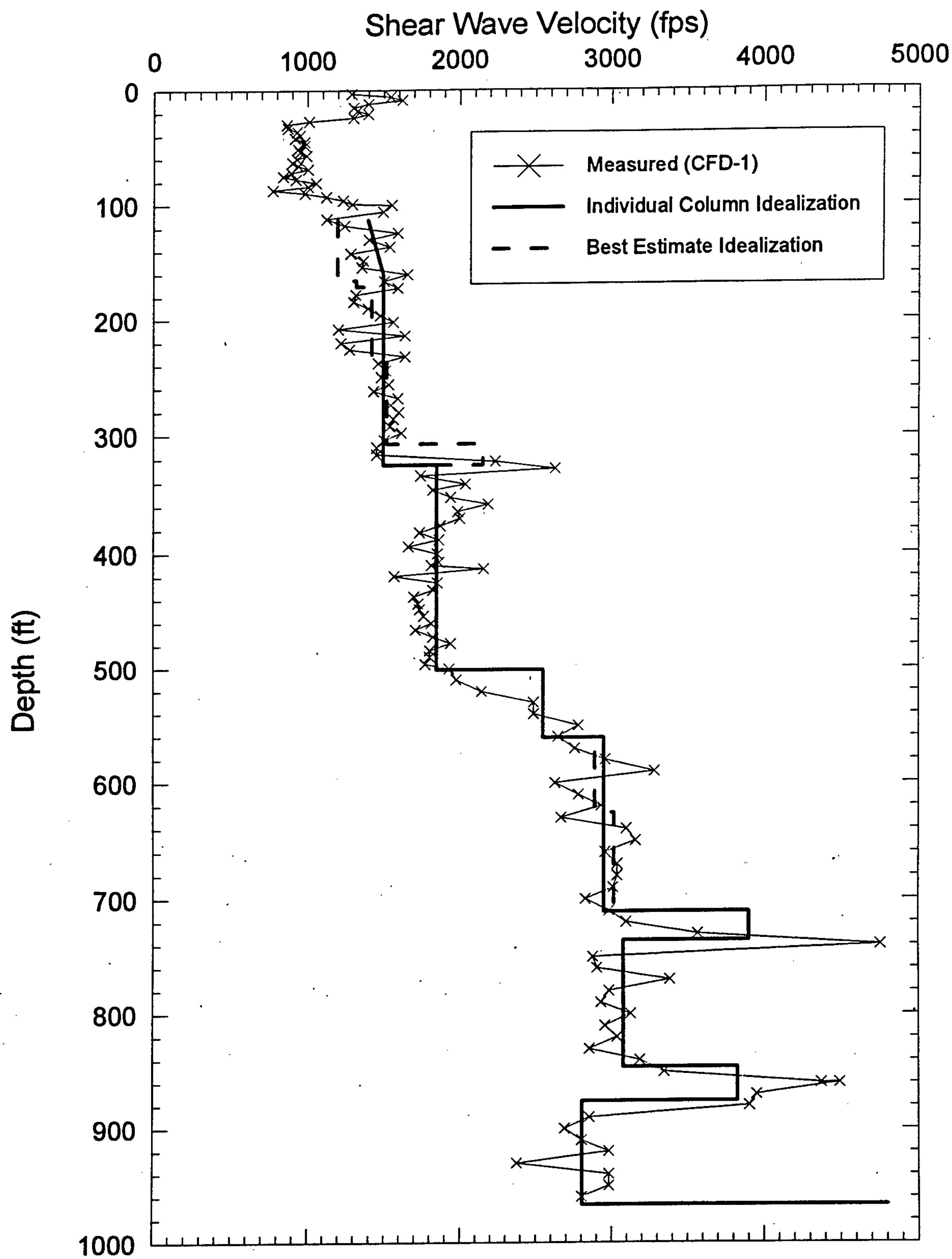


Figure 4.10 Idealized shear wave velocity profile for deep portion of ITP column at SRS

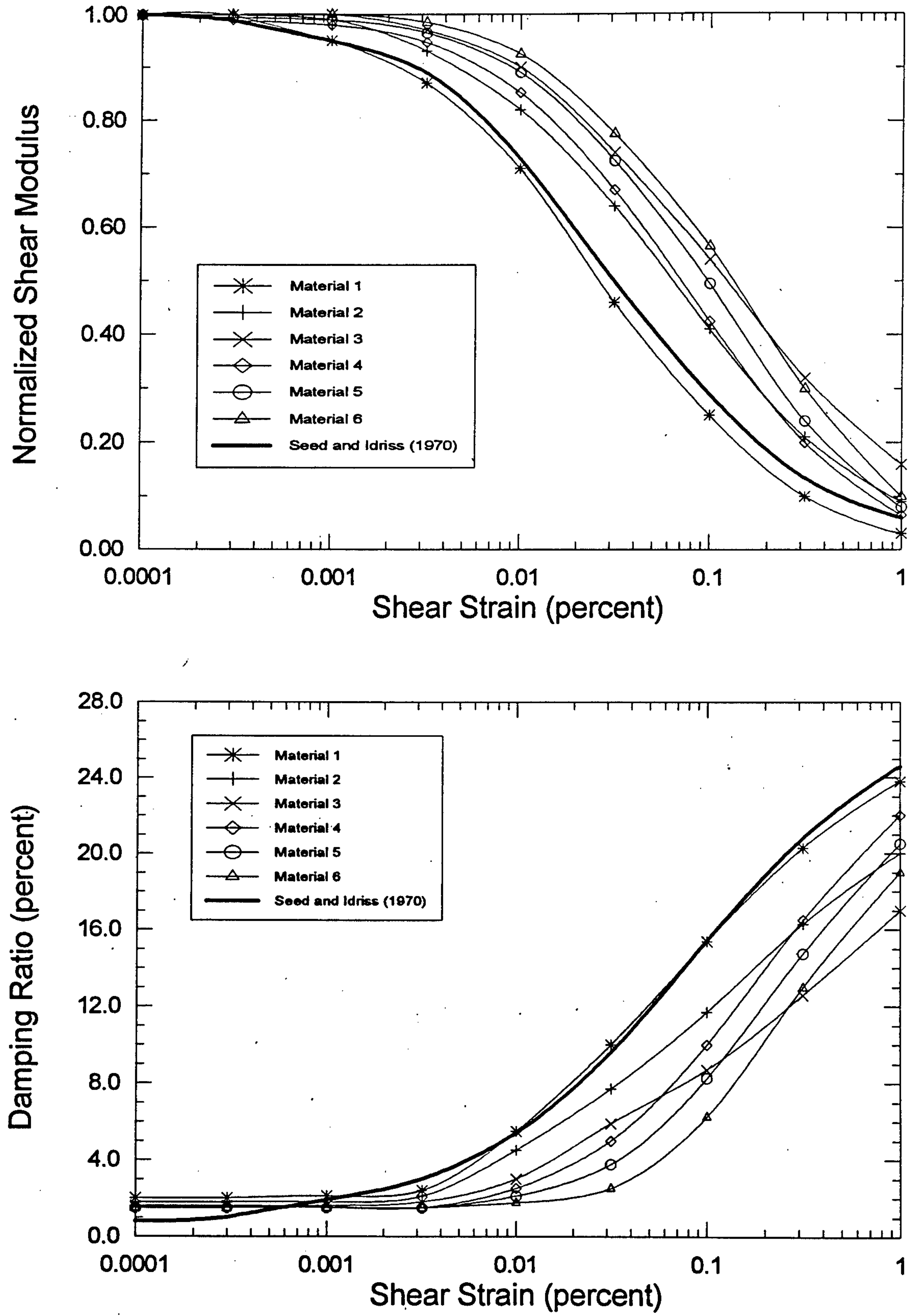


Figure 4.11 Variation in normalized shear modulus and damping ratio with effective shear strain used for site response calculations at ITP of SRS

strain.grf

Analysis

The results from the site response calculations for the four individual columns are shown on Figure 4.12 which differ significantly from the rock outcrop spectrum which is also shown. The upper bound formed from the four columns has four fairly common peaks between 1.5 and 5 Hz with spectral accelerations between 0.17 and 0.20. In general, the ground motions are amplified at frequencies less than 10 Hz. The range in spectral accelerations and spectral ratio is fairly narrow which reflects that the shear modulus is varied over only the upper 15 percent of the column height.

The calculated response using the best-estimate column and the upper and lower bounds to shear modulus are shown on Figure 4.13. The response from best-estimate column closely follows the range from individual columns up to a frequency of about 2 Hz. At greater frequencies, the best-estimate column significantly under predicts (by up to 25 percent) the upper-bound spectral response from the individual columns. The addition of the response from the upper and lower bounds does not enhance the ability to match the spectral peaks at frequencies above 2 Hz. The primary reason for this situation lies in the fact that the average or best-estimate soil column was selected to have essentially a uniform variation of shear wave velocity with depth. The additional amplifications at the higher frequencies are related to the variations in shear wave velocity between soil layers at depth.

Figure 4.14 indicates a comparison of the surface spectra from the four individual columns together with the envelope spectrum obtained from the best-estimate and upper and lower bounds for the uniform average soil column. As can be noted, this envelope still misses capturing the peaks induced from the differences in shear wave velocities for the soil layers in the individual soil columns. These results again indicate that to capture the peaks of the spectra, the best-estimate soil column must be carefully selected so as to properly capture the effects of differences in the soil properties.

The variation in shear modulus from ½ to 2 times the best-estimate values

was also applied to the individual soil columns since the profiles of shear wave velocity for the individual columns did not seem to adequately represent the full range of shear wave velocity. The upper- and lower- bound profiles of shear wave velocity for the individual columns are shown on Figure 4.15. These profiles bound nearly all the measured values and tend to significantly over predict the range of velocities at the upper bound.

The calculated response for the eight columns produced with the upper- and lower-bound profiles are shown on Figure 4.16. The ranges of calculated response for the individual columns are also shown for comparison. The collection of response spectra tend to diverge as the frequency increases with the four lower bound spectra becoming rather constant at about 0.65 g and the four upper bound spectra varying considerably between 0.1 and 0.22 g. This divergence reflects the influence of near-surface modulus on peak ground acceleration. These data suggest that using upper and lower bound variations with a number of individual soil columns and then enveloping the results can lead to extremely conservative estimates of surface response spectra. This conclusion is supported by a comparison with measured variability in soil sites presented in Chapter 5.

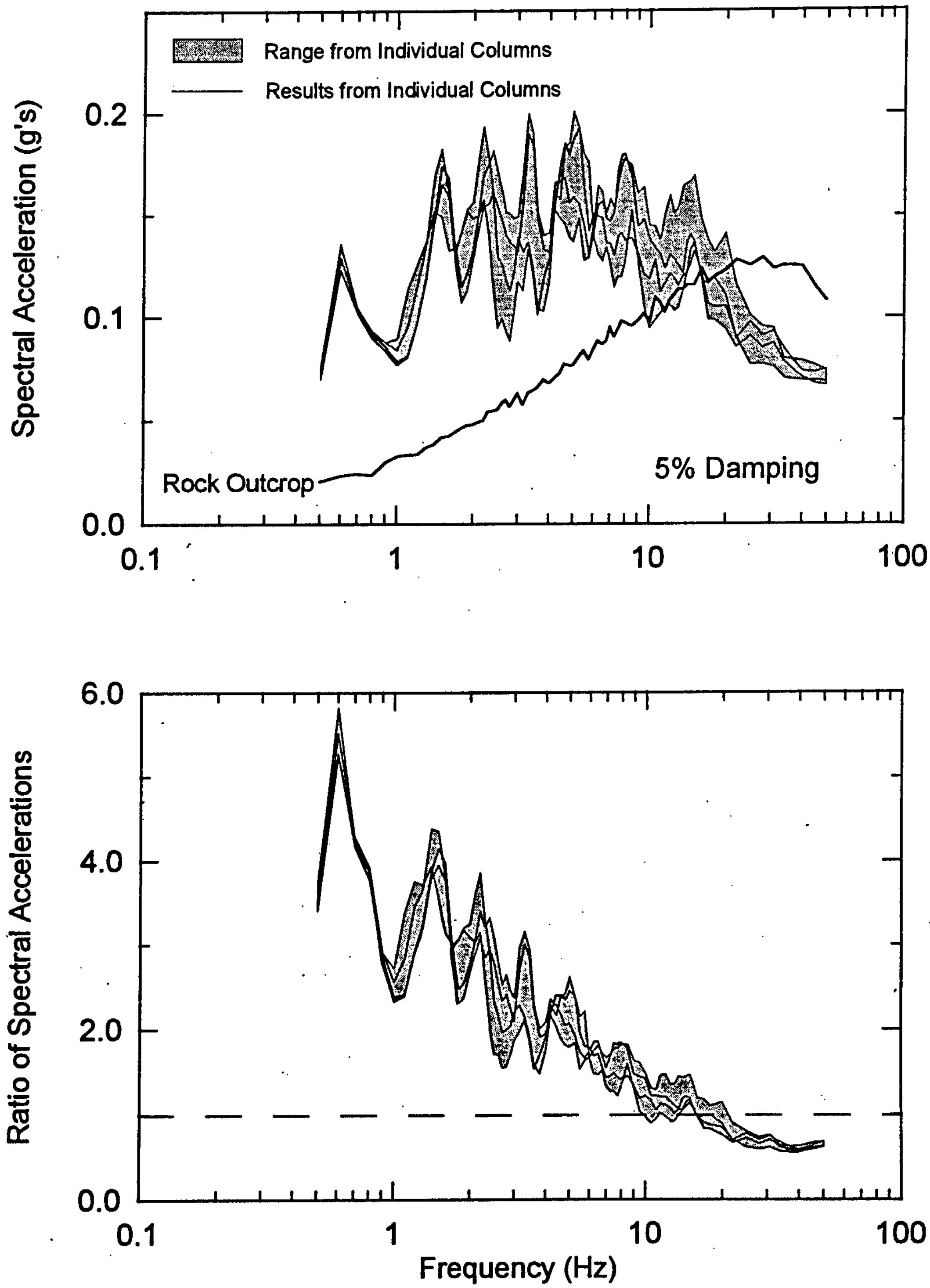
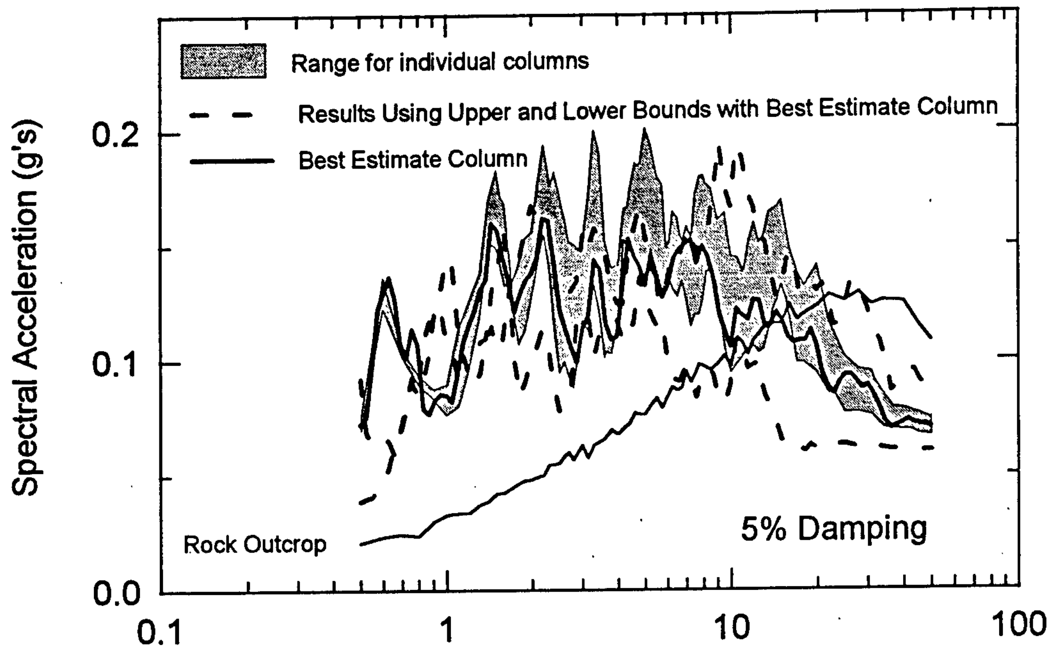


Figure 4.12 Response spectra calculated for individual columns at ITP of SRS

w_sra.grf



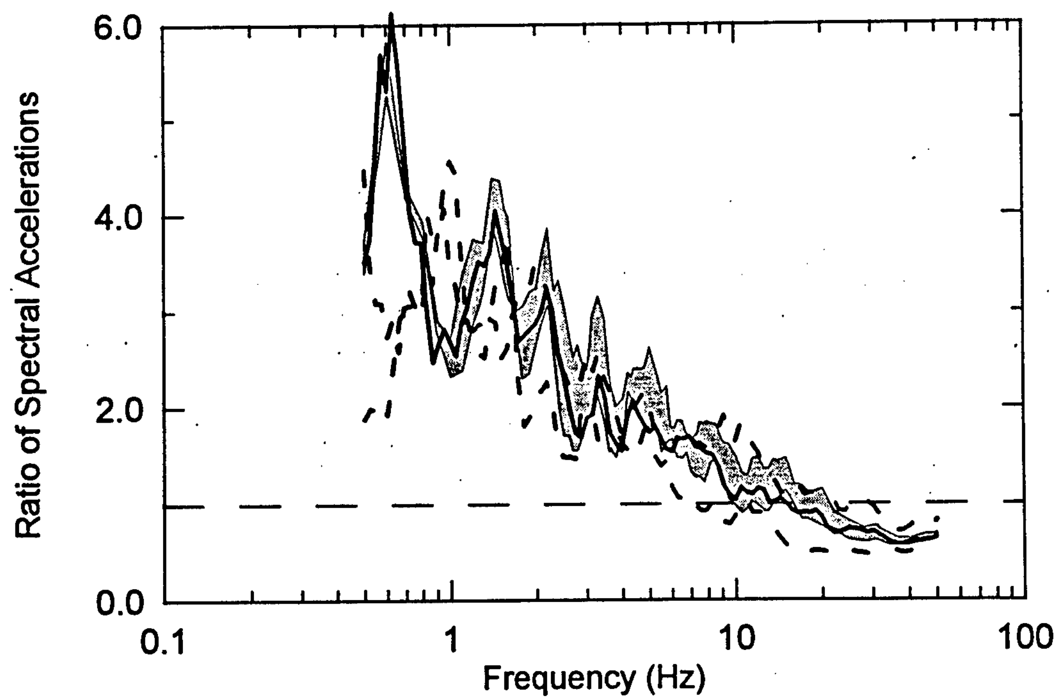


Figure 4.13 Comparison of spectral accelerations at ITP of SRS

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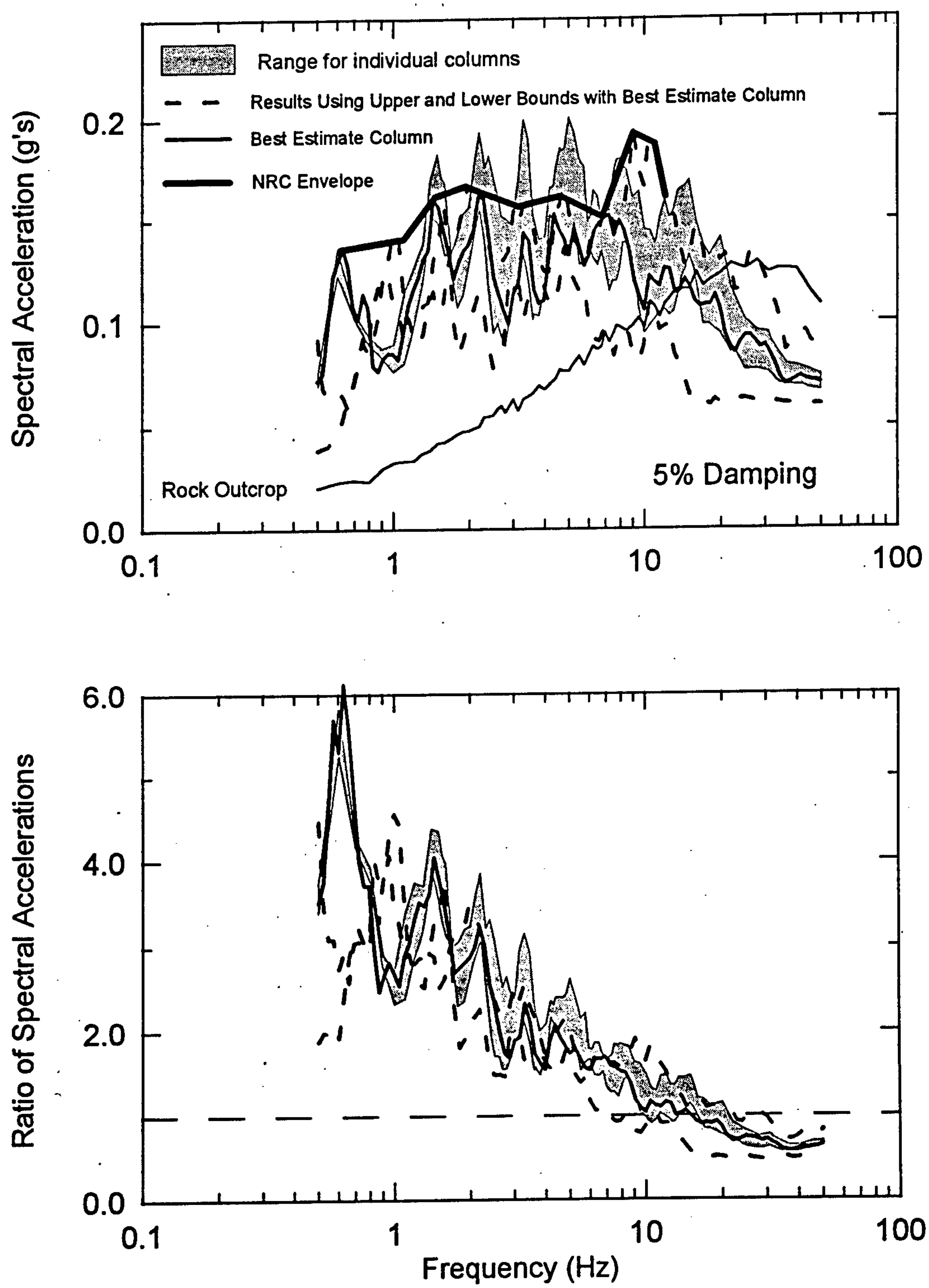


Figure 4.14 Comparison of results at ITP of SRS using NRC envelope

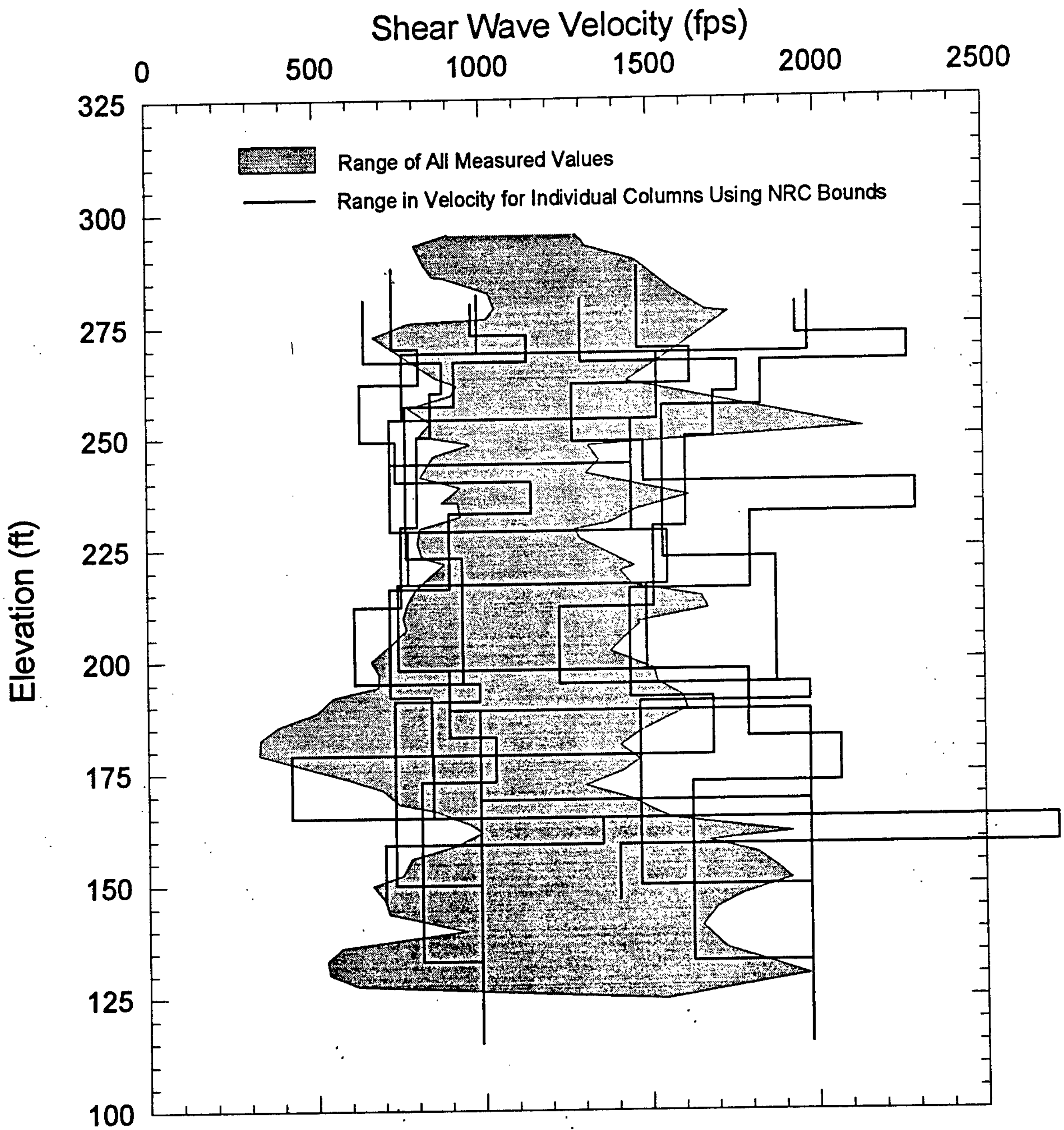


Figure 4.15 Variation in shear wave velocity using upper and lower bounds on individual columns at ITP of SRS

vs_ic.grf

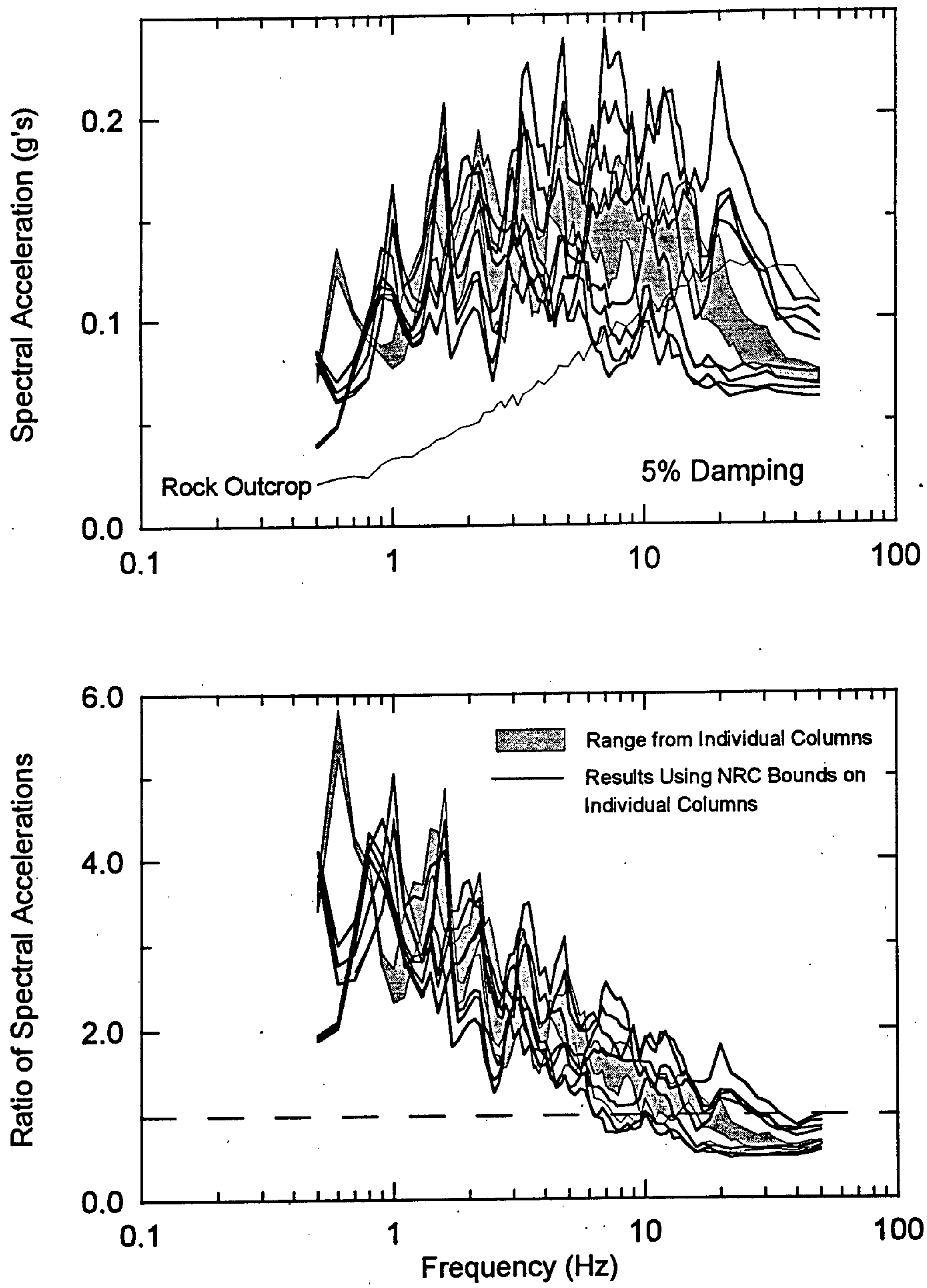


Figure 4.16 Response spectra for individual columns using NRC bounds to shear modulus at ITP of SRS

w_sra_r.grf

5 Estimation of Ground Response Variability

The results presented in the previous two chapters show that wide variations in ground response for a large project site can be calculated using individual columns or best-estimate columns with recommended upper and lower bounds. These data derived from 1-D analysis do not indicate if ground motions can vary that much over a site. Therefore, empirical recordings of seismic ground motion at dense arrays were used to quantify the variability of site response over short distances. In a previous study, Schneider et al. (1992) used dense array recordings to analyze the variability of Fourier amplitude spectra over short distances. Here, the variability of the response spectra is analyzed for a subset of the same data set.

The variability of site response is estimated as follows. Let $Sa_{ij}(f)$ be the average horizontal component acceleration response spectrum for the jth station for the ith earthquake. Let $\Delta Sa_{ijk}(f)$ be the difference between the log spectral values of the jth and kth stations from the ith event:

$$\Delta Sa_{ijk}(f) = Sa_{ij}(f) - Sa_{ik}(f) \tag{5}$$

Let $\sigma(f,\xi)$ be the standard deviation of $\Delta Sa(f)$ where ξ is the separation distance between stations j and k. Assume that $\sigma(f,\xi)$ is independent of the event (but may be magnitude dependent) so that $\Delta Sa(f)$ from different events can be analyzed together. Then σ represents the standard deviation of the difference in site response between two points separated by distance ξ and it is used to quantify the variability in ground response.

Data Set

The largest set of dense-array strong-motion recordings are from the SMART-1 and LSST arrays in Taiwan; however, there are several other dense arrays in California and Japan that have recorded strong motions. In addition, there are several dense arrays that have recorded weak motions.

We considered only arrays with minimum station separations of less than 100 m and obtained recordings from eight dense arrays for use in the analysis. The arrays and their general characteristics are listed in Table 5.1. For this study, the arrays have been grouped only by the general site classes of soil and rock, with four arrays on rock and four on soil. The data sets for each array are summarized in Table 5.2. Five of the arrays have recorded strong motion and three have recorded only weak motion.

Model

As the separation distance, ξ, goes to zero, σ goes to zero by definition. Based on the previous study of variability of Fourier amplitudes by Schneider et al. (1992) and a preliminary analysis of the site response variability, the site response variability for a given frequency and magnitude range is modeled by:

$$\sigma(f,\xi) = c_1(f,M)\left(1 - e^{\xi c_2(f)}\right) \qquad (6)$$

where $c_1(f,M)$ and $c_2(f)$ are constants for each frequency and magnitude range. Using the dense array data, the constants c_1 and c_2 are estimated by regression using a maximum likelihood approach. The resulting values of $c_1(f, M)$ are listed in Table 5.3 and the resulting model for c_2 is:

$c_2 = 0.2f / 3.5$ when $f \leq 3.5$ Hz

$$c_2(f) = 0.2 \qquad \text{when } f > 3.5 \text{ Hz}$$

The site response variability model is plotted as a function of frequency in Figs. 5.1 and 5.2 and as a function of separation distance in Fig. 5.3. The variability is strongly dependent on earthquake magnitude with large magnitude events having less variability than small magnitude events. This result is consistent with recent analyses of large empirical strong motion data bases (Youngs et al. 1993). The variability of site response for soil sites is between 10 and 20 percent for moderate to large magnitude (M > 5) events.

Comparison of Soil Site and Rock Site Ground Response Variability

The majority of the dense array data used in this study is from soil site arrays, however, there is some data from rock site arrays. The variability of ground response on rock and soil sites are shown by the solid and open symbols, respectively, in Figures 5.1 and 5.2. These figures show that the variability of ground response at rock sites is larger than at soil sites. The small magnitude (M = 3.0 to 4.1) rock site variability is much larger than the small magnitude (M = 4.0 to 4.9) soil site variability. The large magnitude (M = 5.2) rock site curve is based on only a single event from the Coalinga array so it is not as robust as the other curves, but it also shows larger variability than the magnitude (M=5.0-5.6) soil site variability in the frequency range of 1 to 7 Hz. One possible source for difference in site response variability on soil and rock is that a small shift in resonance across a site can generate large variabilities in amplitude at a given frequency. In this regard, small changes in layer thickness would produce more predominant shifts in resonance for shallow layers; thus shallow soil sites and rock sites with complex geology would tend to experience the largest amplitude variations.

This difference in soil site and rock site ground response variability has important practical consequences. In site response studies, the variability of the rock ground motion at the base of the soil column is often assumed to be the same as the variability at a rock outcrop. The total variability of ground motion at soil sites is

computed by combining the variability of the soil response with the variability of the input rock motion. With this approach, the variability of ground motion at soil sites would be larger than the variability of the ground motion at rock sites. The dense array data, however, suggest that the opposite is true. Therefore, in a site response study, the variability of the computed soil response should not be simply combined with the variability of free-field rock motions to estimate the total variability of the soil site motion.

Comparison with Calculated Response

The calculated variability in response for the shallow soil and deep soil sites were compared with measured variability to assess the reasonableness of idealization schemes, particularly in applying bounds to shear modulus for sensitivity analyses. The spectra for individual columns and the standard error for the shallow soil site (M7 event) and deep soil site are shown on Figure 5.4 and 5.5, respectively. The standard error sub-plot on the bottom of these figures includes the data shown on Figures 5.1 and 5.2 for the range of magnitudes of 3.0 to 7.8. In addition, another comparison was made using the individual columns plus their results using the upper and lower bounds in shear modulus for the deep soil site as shown on Figure 5.6.

The results shown on Figures 5.4 and 5.5 indicate that the standard errors for the calculated individual column response are similar to the measured variability. The average error between 1 and 20 Hz is relatively constant at about 0.1, independent of the frequency. For the deep soil site, the standard errors match well at frequencies less than 4 Hz and match favorably at frequencies greater than 10 Hz.

The results shown on Figure 5.6 indicate that the standard errors for the calculated individual columns combined with the response using the NRC upper and lower bounds is generally much larger than the measured variability except at frequencies between 3 and 6 Hz. Based on the wide range of spectra and the comparisons with measured values, the NRC range may be appropriate for

evaluations with an best-estimate column but appear to be too severe using a collection of individual columns.

Table 5.1

Dense Array Characteristics (Abrahamson 1993)

Array	Location	Site Class	No. Surface Stations	Spacing (m) Min.	Spacing (m) Max.	Reference
EPRI LSST	Taiwan	Soil	15	3	85	Abrahamson et al. (1991)
EPRI Parkfield	CA	Rock	13	10	191	Electric Power Research Institute (1988)
Chiba	Japan	Soil	15	5	319	Katayama et al. (1990)
USGS Parkfield	CA	Rock	14	25	952	Fletcher et al. (1991?)
Imperial Valley Differential	CA	Soil	5	18	213	Smith, Ehrenberg, & Hernandez (1982)
Hollister Differential	CA	Soil	4	61	256	Salsman and Forshee (1988)
Coalinga*	CA	Rock	7	48	313	Mueller, Sembera, and Wennerberg (1984)
USCS ZAYA*	CA	Rock	6	25	300	Bonamassa et al. (1991)

* Temporary arrays

Table 5.2

Dense Array Data Sets (Abrahamson 1993)

Array	No. of Events	Magnitudes Min.	Magnitudes Max.	Distances (km) Min.	Distances (km) Max.	a_{max} (g)
EPRI LSST	9	3.0	7.8	5	113	0.26
EPRI Parkfield	2	3.0	3.9	13	15	0.04
Chiba	5	4.8	6.7	61	105	0.41
USGS Parkfield	1	3.5		18	45	0.04
Imperial Valley Differential	2	5.1	6.5			0.89
Hollister Differential	1	5.3		17		0.20
Coalinga	6	3.2	5.2	12		0.21
UCSC ZAYA	3	2.3	3.0	9	19	0.03

Table 5.3

Regression Results for Parameter c_1 (Abrahamson 1993)

Freq. (Hz)	Soil M=4.0-4.9	Soil M=5.0-5.9	Soil M=6.0-7.8	Rock M=3.0-4.1	Rock M=5.2
0.7	-	0.095	0.087	-	0.15
1.5	0.12	0.10	0.095	-	0.16
2.5	0.16	0.13	0.12	-	0.21
3.5	0.30	0.19	0.18	0.46	0.26
5.0	0.32	0.20	0.19	0.48	0.38
7.0	0.31	0.20	0.20	0.50	0.26
9.0	0.30	0.20	0.18	0.58	0.16
11.0	0.30	0.20	0.15	0.65	0.14
15.0	0.30	0.20	0.13	0.74	0.14
20.0	0.30	0.20	0.13	0.80	0.14
25.0	0.30	0.17	0.13	0.76	0.14

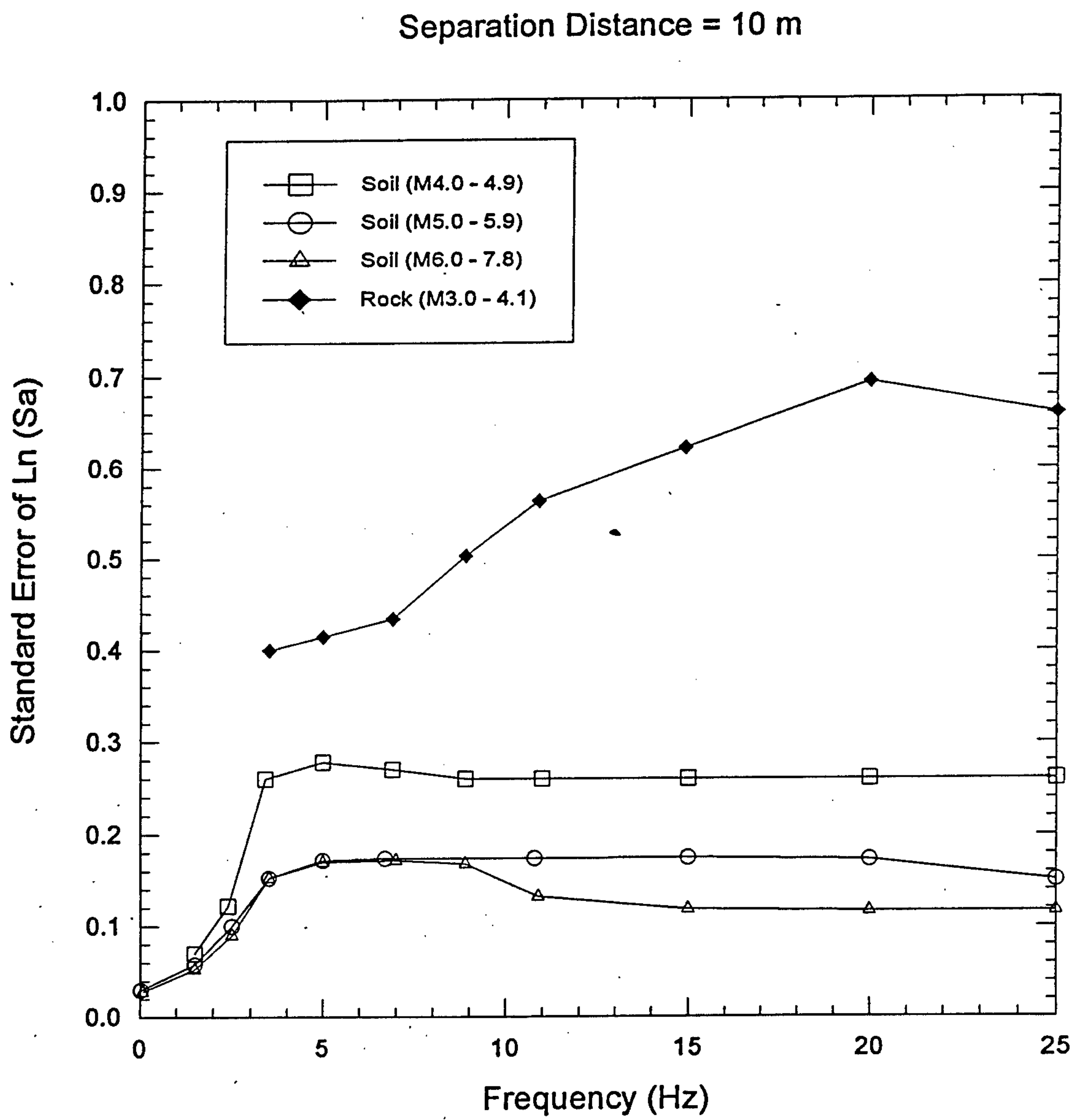


Figure 5.1 Variability of ground response for a separation distance of 10 meters

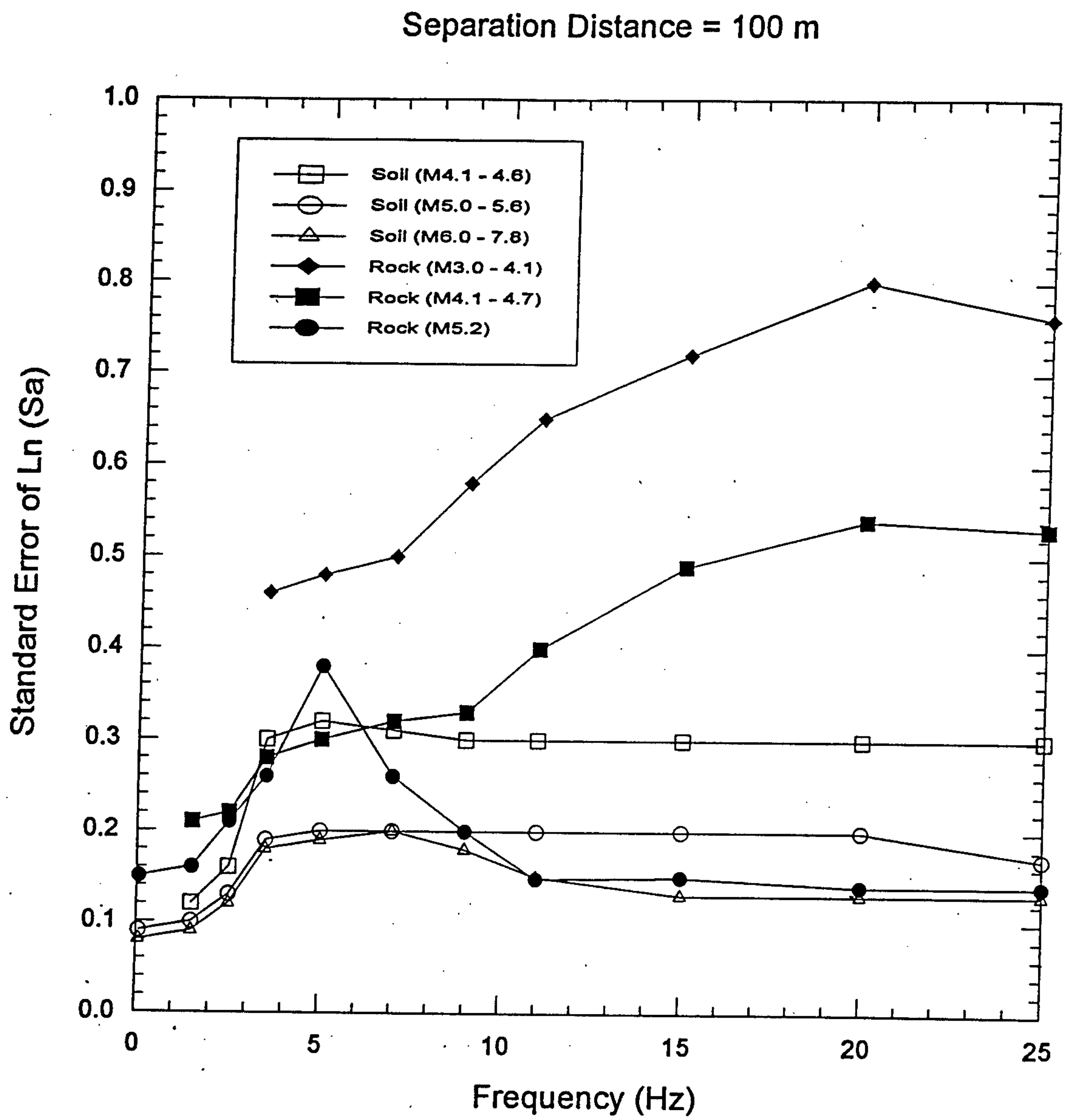


Figure 5.2 Variability of ground response for a separation distance of 100 meters

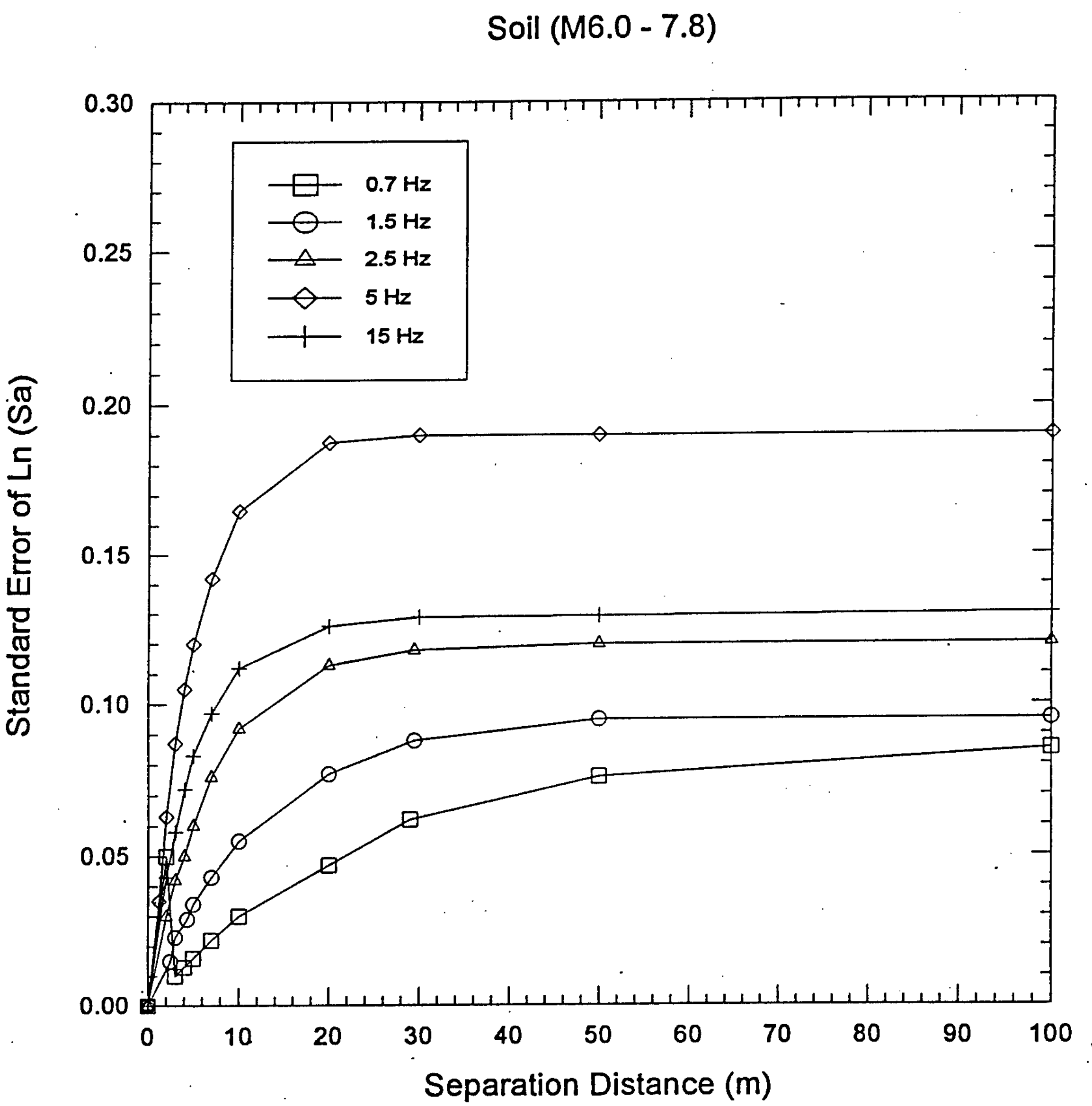


Figure 5.3 Variability of ground response for large magnitude events at soil sites

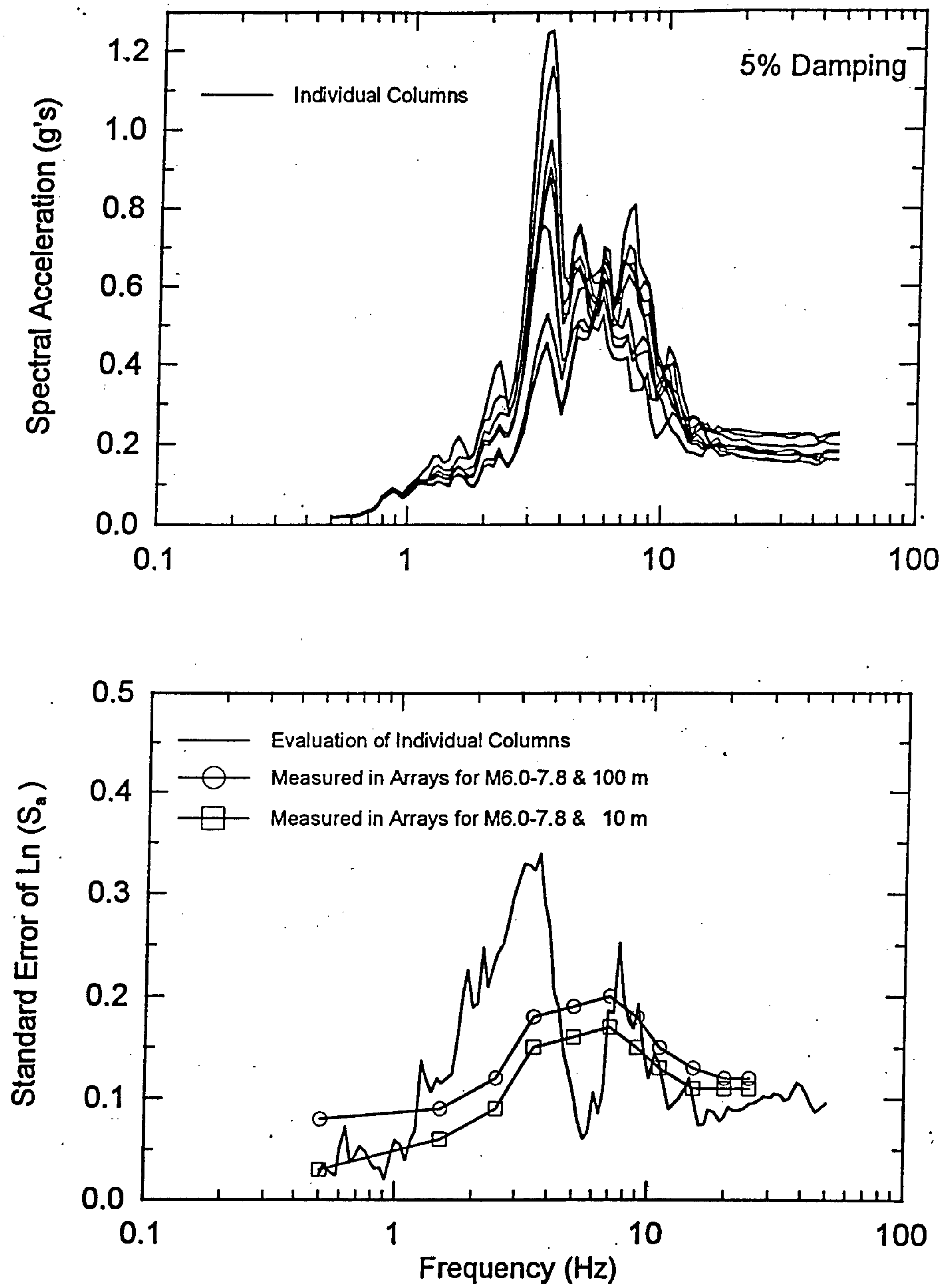
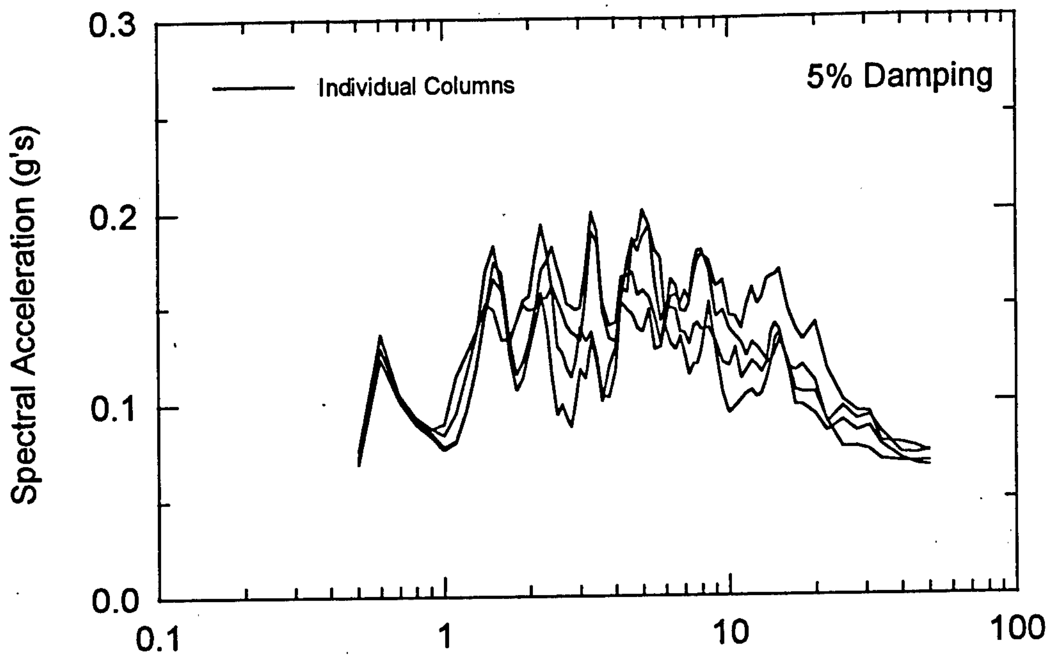


Figure 5.4 Comparison between response at arrays measured during earthquakes and calculated response using individual columns for shallow site at PORTS

w_sra_s3.grf



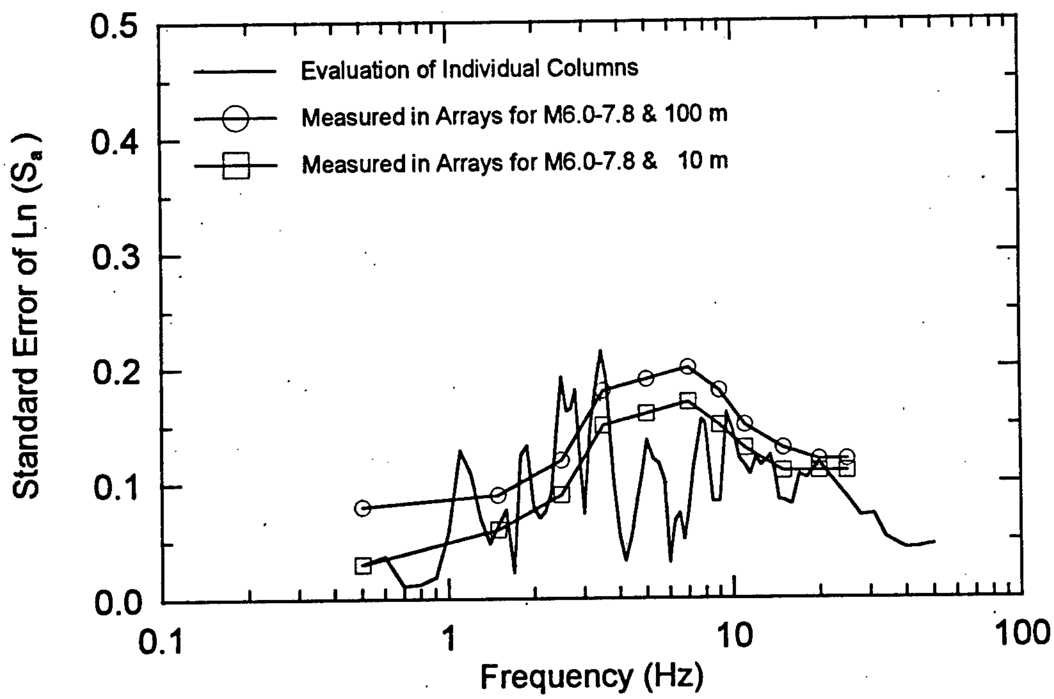


Figure 5.5 Comparison between response at arrays measured during earthquakes and calculated response using individual columns for deep site at ITP of SRS

w_sra_sd.grf

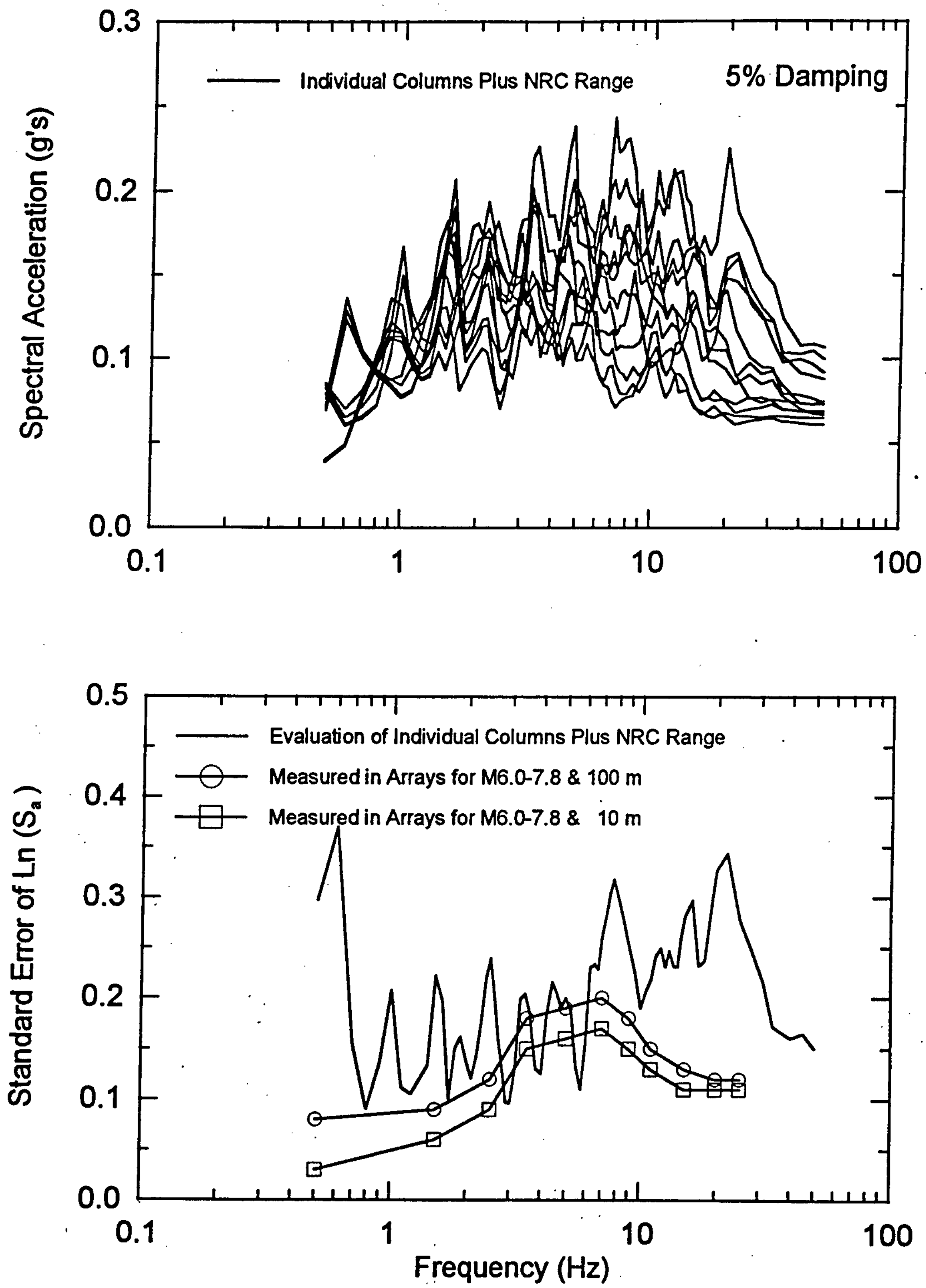


Figure 5.6 Comparison between response at arrays measured during earthquakes and calculated response using individual columns for deep site at ITP of SRS and parametric analysis of shear modulus

w_srs_s2.grf

6 Summary and Conclusions

The results from one-dimensional (horizontal) site response analyses for a shallow project site and a deep project site at two DOE facilities provide a substantive basis to compare the effects of two methods of idealizing soil deposits. These two methods were termed the individual soil column and best-estimate soil column methods for purposes of this study. In general, the two idealization methods were found to generate similar results for the shallow site, despite a relatively large percent variation in depth to rock, and substantially different results for the deep site which was assumed to have a constant depth to rock.

The results of comparisons made herein are insufficient to conclude which idealization method is more appropriate for seismic safety or design evaluations. The best idealization method should be chosen based upon direct comparisons between estimated and measured response at site.

The individual soil column method involves evaluating the variation of soil properties with depth for each small area or boring location at which the geotechnical information, most importantly shear wave velocity, is measured. For large project sites, such as the two sites considered, several sets of such information may be available. An upper bound site response is evaluated from the collection of individual response results.

The best-estimate soil column method involves collectively evaluating the variation of soil properties with depth to produce a single representative soil column. Typically, the best-estimate method involves a parametric evaluation of shear modulus by defining upper and lower bounds and then using upper-bound enveloping of the spectra to account for spatial variability of material properties. The NRC (1989) and ASCE (1986) provide guidance for selecting upper- and lower- bound shear moduli for these analyses.

The comparison between idealization methods for the shallow site was performed using nine soil columns and three measured WUS earthquakes records representing three combinations of magnitude and distance. The peak ground accelerations were 0.08 and 0.12 g and the frequency content was generally broad-banded. The peak values of spectral acceleration and ranges produced by the nine columns were found to be relatively independent of the earthquake and to be amplified over the rock outcrop spectra for frequencies analyzed.

The upper bound site response of the best-estimate column and variations in the profile of shear modulus defined by enveloping for the shallow site matched well for all three earthquakes with the upper bound formed from the collection of individual responses. This favorable comparison exists despite a fairly wide variation in soil column height (20 to 61 ft); the best-estimate column assumed a 42 ft column height. Conversely, the lower bounds formed by the two methods differed significantly. Use of spectra enveloping for the best-estimate upper bounds provided a more consistent lower bound and a slightly improved comparison at the upper bound. An even closer comparison among upper-bound results was produced by using a Monte Carlo scheme for the random selection of shear modulus for each layer about the best-estimate values and a statistical analysis of results.

The comparison between idealization methods for a deep site (about 970 ft) was performed using four soil columns and one synthetic earthquake record representing an SEUS earthquake source with $M_w = 7.5$ and a peak acceleration of 0.055 g at the rock outcrop. The four soil columns were based on individual seismic velocity profiles for about the upper 150 ft and a single seismic velocity profile for the lower 820 ft. The calculated responses among the four individual columns were similar, each having several comparable peak values of spectral acceleration.

The upper bound site response of the best-estimate column and variations in the profile of shear modulus defined by NRC for the deep site did not compare well with the upper bound formed from the collection of individual responses, even when spectra enveloping was used with the best-estimate results. The best-estimate column method significantly under predicted the range of response estimated using the

individual columns, particularly at higher frequencies.

The primary factor causing the difference between the results for the individual soil column and best-estimate soil column methods for the deep site is the poor representation of impedance contrasts with depth. The measured data and the individual column representations, shown on Figure 4.6, have significant impedance contrasts, similar to the measured data, as shown on Figure 4.7. The best-estimate column shown on Figure 4.9, however, has very small contrasts, the product of "averaging" numerous values at each depth.

Although the application of the NRC upper-bounds on shear modulus (two times the best-estimate) plus enveloping enhances the ability of the best-estimate column method to represent the upper bound range of response calculated with the individual columns, the application of this same upper bound to each individual soil column appears to be overly conservative. This is supported by comparisons with measured spatial variability during previous large magnitude earthquakes (M6.0-7.8). These limited findings suggest that the individual column method inherently incorporates sufficient variability to represent the range of soil conditions. At a maximum, variability associated with potential measurements errors can be introduced to each column.

An evaluation of measured earthquake response using ground surface arrays indicates that the variability at rock sites is greater than the variability at soil sites and that spectral accelerations can vary considerably over distances of 10 m. This important finding suggests that soil sites provide a more uniform filtering of the seismic waves. The results using the shallow site idealization with three earthquake records showed that the calculated response using three earthquake records for different magnitudes were fairly similar.

The results of this study suggest that certain aspects of the two methods for developing soil columns for one-dimensional site response may be very important under certain circumstances. The information available does not provide enough evidence to show which of the two methods is a better representation of what can be

expected in the field. However, the initial findings presented suggest that the best-estimate column, together with enveloping for the upper-bound response, can lead to under prediction of peak spectral accelerations.

7 Recommended Procedures

Although this study was limited in scope to a comparative evaluation of results from only two (DOE) sites, preliminary recommendations are provided to augment existing practice and procedures. These preliminary recommendations relate to the use of each idealization method and not to which method is more appropriate. Furthermore, these preliminary recommendations are considered to be appropriate only for relatively large sites that have a significant amount of seismic geophysical data from which to idealize soil columns for site response analyses, similar to the two sites analyzed for this study.

The individual soil column method, as outlined in this report, appears to provide a reasonable upper bound of calculated responses. Higher priority should be given to the use of shear modulus profiles with large measured impedance contrasts and profiles defining the upper and lower bound for the site. If additional variability is desired to represent uncertainty in material properties, the authors suggest that the profile of shear wave velocity or shear modulus should be varied randomly within measurement error limits (on the order of 5 to 15 percent).

The best-estimate soil column method may be sensitive to the impedance contrasts in the best-estimate profile of shear modulus. Therefore, the authors recommend that careful consideration be given the "averaging" process, particularly for sites at which the idealized layers are relatively thin. One option may be to vertically shift profiles of shear wave velocity to align layers with similar velocity prior to averaging.

The NRC and ASCE recommended parametric bounds to shear modulus are substantial and enveloping (NRC) introduces even more conservatism. Unlike the individual column method, these bounds are independent of the known site characteristics, but inappropriate including material variability. These bounds appear to be appropriate for the best-estimate soil column method but inappropriate for the

individual soil column method. For the best-estimate soil column method, the authors recommend that these bounds be applied through Monte Carlo simulations of material variability independently for each layer and statistical evaluation at the two standard deviation level. Enveloping would not be required. This procedure will allow for variability in impedance contrasts which have been shown to be important.

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13. ABSTRACT *(Maximum 200 words)*

This report addresses the effect of different idealization schemes on calculated dynamic site response analyses. The two methods compared were the "individual column" and the "best estimate" methods. The "individual column" method uses several unique individual soil columns, each developed from specific subsite data. The "average column" method involves averaging information from each subsite into one idealized representation. Use of the "average column" method typically involves a parametric analysis of model parameters, most importantly, shear modulus. This report compared these methods for two projects site: DOE Portsmouth Gaseous Diffusion Plant and DOE Savannah River Site. The results of the two idealization schemes are also compared with measured variations of ground motion response during earthquakes. In general, the two idealization schemes produced similar results for the shallow site, despite a relatively large variation in depth to rock, and substantially different results for the deep site which was assumed to have a constant depth to rock.

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